ALA WAI CANAL PROJECT
O'AHU, HAWAI'I

DRAFT FEASIBILITY STUDY REPORT WITH INTEGRATED ENVIRONMENTAL IMPACT STATEMENT

APPENDIX A
HYDROLOGIC & HYDRAULIC ANALYSIS AND ENGINEERING DESIGN

Existing Without-Project Hydrologic Appendix

Ala Wai Canal Project

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Appendix A1

Table of Contents

List of Figures

List of Tables

Abbreviations and Acronyms

A	Ala Wai (designating sub-watershed)
BWS	Board of Water Supply
City	City and County of Honolulu
cfs	cubic feet per second
CN	curve number
USACE	United States Army Corps of Engineers
DLNR	State of Hawaiʻi, Department of Land and Natural Resources
FEMA	Federal Emergency Management Agency
GIS	geographical information system
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HEC-SSP	Hydrologic Engineering Center-Statistical Software Package
HEC-GeoHMS	Hydrologic Engineering Center Geospatial Hydrologic Modeling Extension
HEC-GeoRAS	Hydrologic Engineering Center Geospatial River Analysis System Extension
HSG	hydrologic soil group
IDF	Intensity-duration-frequency
ID	Identification number
J	junction
K	Makiki (designating sub-watershed)
LiDAR	light detection and ranging
M	Mānoa (designating sub-watershed)
MP	Mānoa-Pālolo (designating sub-watershed)
MSL	Mean sea level
MWP	Mānoa Watershed Project
NOAA	National Oceanit and Atmospheric Agency
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
P	Pālolo (designating sub-watershed)
SCS	Soil Conservation Service
T_c	time of concentration
TR-55	Technical Release 55
UHM	University of Hawaiʻi at Mānoa
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
W	Waikīkī (designating sub-watershed)

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Executive Summary

Introduction

In 2001, the United States Army Corps of Engineers (USACE) recommended flood mitigation and ecosystem restoration measures for the Ala Wai Watershed, located on the southeast sector of the island of O'ahu, Hawai'i. As part of this larger goal, USACE contracted Oceanit to develop a Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) for a range of potential storms in the Ala Wai Watershed. HEC-HMS is the USACE hydrologic model. The purpose of this study was to estimate peak flow discharges at particular drainage junctions in the Ala Wai Watershed corresponding to the following storm return periods: 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year. These storm return periods correlate to storm chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent, respectively.

Purpose

Whereas this study focuses on the HEC-HMS model, this study uses a total of five different methods to estimate peak flow discharges throughout the Ala Wai Watershed for potential storms ranging in duration and intensity. Estimated peak flow discharges are based on the existing conditions of the Ala Wai Watershed's sub-watersheds of Makiki, Mānoa, and Pālolo valleys; Mānoa- Pālolo and Ala Wai Canals; and Waikīkī. Discharge at junctions of interest throughout these sub-watersheds was studied. Oceanit modeled storms using both rainfall-runoff and peak flow frequency methods for a range of storm scenarios, as follows. The study (1) researched and collected relevant hydrologic data; (2) constructed and calibrated both rainfall-runoff and peak flow frequency hydrologic models; and (3) weighted and compared the results from these models to arrive at estimated peak flow discharges.

Study Area

The Ala Wai Watershed encompasses a drainage area of 10,400 acres (16.2 square miles) of area that are economically significant and densely populated. The existing conditions throughout the Ala Wai Watershed are relevant to its hydrologic analysis, including the character of the watershed's overall climate, topography, geology, vegetation, land use and cover, and water resources. Hawai'i's high moisture, orographic rainfall, and northeasterly trade winds create wet conditions in the upper Ala Wai Watershed. The topography of the upper Ala Wai watershed is relatively steep and stony that, in combination with heavy rainfall, provides conditions prone to flash flooding. The lower Ala Wai watershed has finer well-drained soil, but much of it is urbanized, meaning its terrain surfaces are impervious. In terms of streams, the Makiki, Mānoa, and Pālolo streams drain their respective sub-watersheds. Mānoa and Pālolo streams combine to form the Mānoa-Pālolo Canal that empties into the Ala Wai Canal. Runoff and drainage from Waikīkī empties into the Ala Wai Canal as well.

Data Collection Procedure

Data collection for hydrologic analysis included rainfall gage data, stream flow gage data, records of historical storms, maps of storm drainage systems, geospatial data, and field surveys observations. Storms that occurred on December 17–18, 1967; October 30, 2004; and March 31, 2006 were used to calibrate the HEC-HMS model. The City and County of Honolulu drainage maps and University of Hawai'i's utility maps were used to determine the existing storm drainage system. Geospatial information, including LiDAR data and aerial maps established terrain roughness characteristics and stream channel cross sections. Rainfall data was extrapolated to be converted into intensity-duration-frequency (IDF) curves, illustrating rainfall intensities according to their duration.

Hydrologic Analysis Procedure

Hydrologic analysis of sub-watersheds of the Ala Wai Watershed predicted from the application of five hydrologic modeling methods: the HEC-HMS model, USGS regression method, City and County of Honolulu drainage standards Plate 6, Federal Emergency Management Agency Flood Insurance Study, and the HEC Statistics Software Package (SSP). The HEC-HMS model of the Ala Wai Watershed was the focus of this report, and the results from this model were relied on more than other methods.

SCS curve number Loss Method was applied and Clark Unit Hydrograph transform method was applied for non-urbanized areas, and the Kinematic Wave Transform Method was used for urbanized areas. The Ala Wai Canal was assumed to be a reservoir for the purposes of this study because of backwater effects that are possible in the mouth of Ala Wai Canal. Also, according to the TR-55 method, the water flow path was separated into three portions: sheet flow, shallow concentrated flow, and channel flow, which are summed to calculated time of concentration. Manning's *n* values were selected for the land surface characteristics for the Ala Wai Watershed. Curve number calculations were established according to the hydrologic soil group.

Results

Final "best" peak flow discharges were determined by comparing the various derived discharge-frequency curves graphically and by the accuracy or uncertainty of each method. Table ES-1 shows the results of peak flows discharges at the mouth of the Ala Wai Canal.

Peak Flow Discharges at Mouth of Ala Wai Canal								
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
Methodology	**Peak flow discharge (cubic feet per second)**							
HEC-HMS (original Dec 2008)	6,000	10,100	13,390	15,190	16,740	17,670	18,690	20,480
Plate 6						22,500		
FEMA			13,700		23,000	28,200		36,200
HEC-HMS (updated Nov 2010)	8,080	11,900	14,400	16,000	17,800	19,100	20,700	22,200
Final Used (November 2010)	**6,000**	**11,500**	**13,500**	**16,000**	**18,000**	**19,500**	**20,500**	**22,000**

Table ES-1. Peak Flow Discharges at Mouth of Ala Wai Canal (Updated November 2010)

1 Introduction

1.1 Background

In 2001, the United States Army Corps of Engineers (USACE) recommended flood mitigation and ecosystem restoration measures for the Ala Wai Watershed, located on the southeast sector of the island of O'ahu, Hawai'i. These measures constitute the Ala Wai Watershed Project that encompasses a drainage area of approximately 10,400 acres of the valleys of Makiki, Mānoa, and Pālolo, and low-lying areas of Mō'ili'ili, McCully, and Waikīkī. These areas are economically significant and densely populated, and many have high potential for flooding. Historically, floods have occurred in the Mānoa, Makiki, and Mō'ili'ili areas due to quick concentration of storm waters that overwhelms the drainage system capacities. Depending on a storm's intensity and duration, the steep slopes of the upper Ala Wai Watershed can create flood conditions due to its steep slopes and impervious surfaces from urbanization. In the past, such as during the severe storm of October 30, 2004, flash flood waters with accumulated debris have caused significant property damage to residential, commercial, and public land (Belt Collins 1998).

Storm runoff in these areas flows through drainage systems that ultimately empty into the Ala Wai Canal. In turn, the Ala Wai Canal flows into the Pacific Ocean. The Ala Wai Canal was constructed in the 1920s, and has experienced heavy sedimentation and economic degradations since its inception (Belt Collins 1998). The proposed flood mitigation measures for the Ala Wai Watershed Project must be based on the best hydrologic and hydraulic data available.

USACE contracted Oceanit to conduct hydrologic analysis for a range of potential storms in the Ala Wai Watershed. This hydrologic study uses five different methods to estimate peak flow discharges throughout the Ala Wai Watershed for potential storms ranging in duration and intensity. Best available predictions are based on the existing conditions of the Ala Wai Watershed's sub-watersheds of Makiki, Mānoa, and Pālolo valleys, Ala Wai Canal, and Waikīkī. Also, the existing conditions of junctions along the Mānoa-Pālolo Canal were considered because of the canal's crucial position as a drainage channel between Mānoa-Pālolo and the Ala Wai Canal, where it empties. Oceanit was directed to model storms using both rainfall-runoff and peak flow frequency methods for a range of storm scenarios, as follows.

1.2 Purpose and Scope

The purpose of this study was to estimate peak flow discharges at particular drainage junctions in the Ala Wai Watershed corresponding to the following storm return periods: 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year. These storm return periods correlate to storm chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent, respectively. The study's scope is solely hydrologic and encompasses the Ala Wai Watershed's sub-watersheds of Makiki, Mānoa, and Pālolo valleys, Ala Wai Canal, and Waikīkī. The study also examines the junctions along the Mānoa-Pālolo Canal.

1.3 Methodology

This hydrologic study provides estimated peak flow discharges for a range of storms for particular junctions throughout the Ala Wai Watershed by applying five hydrologic methods as appropriate and necessary. The following were completed in this study: (1) relevant hydrologic data was researched and collected; (2) rainfall-runoff models were constructed and calibrated; (3) peak flow discharges based on rainfall intensity-frequency-duration curves were modeled; and (4) these peak flow discharges were weighted and compared to arrive at final results that represent the *best* estimated peak flow discharges.

First, research on the overall existing conditions in the Ala Wai Watershed study area was conducted. Section 2.1 describes these overall existing conditions, and then Sections 2.2 through 2.4 detail the existing conditions in each sub-watershed. Conditions that were necessarily evaluated for hydrologic modeling included the slope, character, elevation, vegetative coverage, acreage, and use of the sub-watershed lands. Many of these conditions were evaluated from review of existing literature, gathering of geospatial data, and inspection during field visits. This data collection is documented in Section 3.5. Sub-basins within each sub-watershed were delineated using the geospatial data (see Section 3.6). Also, Manning's *n* values, which describe land cover and roughness, were selected (see Section 4.1.5). The existing conditions of drainage systems in the study area were primarily collected from the City and County of Honolulu's Storm Drainage System Maps (Section 3.4), and were confirmed during field visits. Primarily, drainage junctions of interest in the Ala Wai Watershed were determined from evaluating the existing drainage facilities.

Second, potential storm rainfall amount determinations were extrapolated from historic rainfall data. The storm rainfall amounts that were the input for the hydrologic model are considered the meteorological model. The rainfall and stream flow data were collected from rain gage and stream flow gage records as available for the study area (see Sections 3.1 through 3.2). Records from three severe storms were collected and later used to calibrate the hydrologic model (see Section 3.3). Rainfall amounts that constitute the frequency storms in the meteorological model were gathered from a study entitled "Rainfall Frequency Study for Oahu" (Giambelluca 1984) known commonly as Report R-73. Rainfall amounts were gathered from Report R-73 for the storm chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent. Intensity-Duration-Frequency curves were established for input into the model.

Third, five methods were used to model the Ala Wai Watershed's hydrology. The rainfall-runoff method used was USACE's Hydrologic Engineering Center–Hydrologic Modeling System (HEC-HMS). The peak flow frequency methods used were the United States Geological Survey (USGS) regression equations, the City and County of Honolulu (the City) Plate 6 storm drainage standards, the Federal Emergency Management Agency (FEMA) Flood Insurance Study for the City and County of Honolulu (2004), and Hydrologic Engineering Center–Statistical Software Package (HEC-SSP). The fourth step in this study was, depending on the data available, applying these methods for each sub-watershed or junctions if available for the range of potential storms: chance exceedance probabilities of 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent. The methods used for each junction (by sub-watershed) are shown in Table 1-1 and designated by a checkmark.

Junction	Drainage area (mi^2)	HEC-HMS	USGS Regression Equations	FEMA-FIS	C&C-Plate 6	HEC-SSP
MAKIKI						
JK1	2.33	√	√		√	
JK2	2.49	√	√	√	√	
JK3	2.89	√			√	
MANOA						
JM8	5.97	√	√	√	√	
PALOLO						
JP1	1.15	√	√		√	√
JP2	2.94	√	√		√	
JP3	3.62	√	√	√	√	√
JP4	4.07	√	√		√	
MANOA-PALOLO						
JMP1	10.04	√	√		√	
JMP2	10.34	√	√	√	√	√
JMP3	10.68	√			√	
ALAWAI						
Mouth of Ala Wai Canal	16.22	√		√	√	

Table 1-1. Methods Used by Sub-Watershed Junction

J = junction; K = Makiki; M = Mānoa; P = Pālolo; MP = Mānoa-Pālolo; and mi = miles. A checkmark indicates a method that was used for a particular junction or outlet.

1.3.1 HEC-HMS Analysis

The HEC-HMS model was the primary method of this study. The HEC-HMS method is a precipitation-runoff process model that requires three components including a basin model, a meteorological model, and a control model. The basin model layout was created according to sub-basin delineation and junctions of interest. For the purposes of this study, sub-watershed refers to the larger areas of Makiki, Mānoa, Pālolo, Ala Wai Canal, and Waikīkī; the term "sub-basin" refers to the smaller sub-watersheds within these sub-watersheds to avoid confusion. Also the term "sub-basin" is commonly accepted for the HEC-HMS model delineation of small drainage areas.

1. **Basin Model:** Under the basin model, Ala Wai Watershed was divided into 38 sub-basins. The SCS loss method and Clark Unit Hydrograph transform methods were applied for upper Makiki, Mānoa, and Pālolo valleys because these areas are considered non-urban. The Kinematic Wave Transform Method was applied for the lower Makiki ,Ala Wai Canal, and Waikīkī areas because these areas are considered urban. Selected stream flow routing methods included the Muskingum-Cunge method to account for the peak flow attenuation and the Modified Puls method to account for the backwater effects for reaches collected in the Ala Wai Canal. Ala Wai Canal was modeled as a reservoir. Several basin models were created based on the calibration and determination purposes.

2. **Meteorological Model:** A meteorological model was used to specify how precipitation would be generated for each sub-watershed in the selected basin model. For calibration purposes, hyetographs were used based on the gage weights. For predictive purposes, the frequency storms were used to produce synthetic flood events, according to exceedance probabilities.

3. **Control Model:** A control model was used to set the computation parameters. This study used a five-minute time interval for all computations.

1.3.2 Peak Flow Discharge Results

Ultimately, all five of these accepted hydrologic methods offer the *best* estimated peak flow discharges at particular junctions through Ala Wai Watershed for a range of potential storms. Available results were first weighted by accuracy or uncertainty of method, and then plotted on log-probability graph paper. Selection was completed for a best fit curve function for the peak flow discharge frequency curve at each junction of interest. Final peak flow discharges are presented in Section 5.

1.4 Acknowledgements

The USACE project managers for this study were Mr. Derek Chow and Ms. Cindy Barger. The hydrologic analysis technical team consisted of Mr. Michael Wong with the USACE and Mr. Simon Li of Oceanit. Oceanit's project manager was Dr. Dayananda Vithanage with assistance from Mr. Jay Stone and Ms. Joanne Hiramatsu. Oceanit's technical staff included Ms. Frances Ajo, Mr. Robert Bourke, Mr. Kevin Gooding, Mr. Jonathan Levy, and Mr. Manabu Tagomori. Thank you to the National Climatic Data Center, Board of Water Supply (BWS), and USGS for providing rain and stream gage data pertinent to this study.

2 Study Area Description

The Ala Wai Watershed contains five sub-watersheds that are addressed in this study: Makiki, Mānoa, Pālolo, Ala Wai Canal, and Waikīkī. The Mānoa-Pālolo Canal is also addressed in terms of its drainage junctions. Section 2.1 describes the existing conditions throughout the Ala Wai Watershed, including the overall climate, topography, geology, vegetation, land use, and water resources. These conditions are similar in each of the Ala Wai sub-watersheds that are described in Sections 2.2 through 2.6.

2.1 Ala Wai Watershed

The subject of this hydrology study is the Ala Wai Watershed, which is located on the southeastern sector of the island of Oʻahu, Hawaiʻi as shown in Figure 2-1. The watershed encompasses 10,378 acres, or 16.215 square miles. The Ala Wai Watershed stretches from the Koʻolau Mountains at Puʻu Kōnāhuanui's peak (3,105 feet) down through the three urban valleys of Makiki, Mānoa, and Pālolo, to the low-lying areas of McCully, Mōʻiliʻili, and Waikīkī. Storm runoff in the watershed flows through numerous drainage systems in these areas and ultimately empties into the Ala Wai Canal. The three major sub-watersheds that constitute the Ala Wai Watershed are Makiki, Mānoa, and Pālolo; all three of these sub-watersheds are valley systems of economic significance and dense population. The Makiki, Mānoa, and Pālolo Streams receive flows from each of these valley systems, respectively (see Figure 2-2). Another Ala Wai sub-watershed is at the confluence of the Mānoa and Pālolo Streams, referred to as the Mānoa-Pālolo Canal, which empties storm water runoff into the Ala Wai Canal between the Ala Wai Golf Course and ʻIolani School. The area surrounding the Ala Wai Canal and the adjacent tourist area of Waikīkī comprise another sub-watershed. These major sub-watersheds are shown in Figure 2-2. (According to the existing conditions, sub-basins are delineated within each sub-watershed, and these sub-basin delineations are presented in Section 3, and shown in Figure 3-4.)

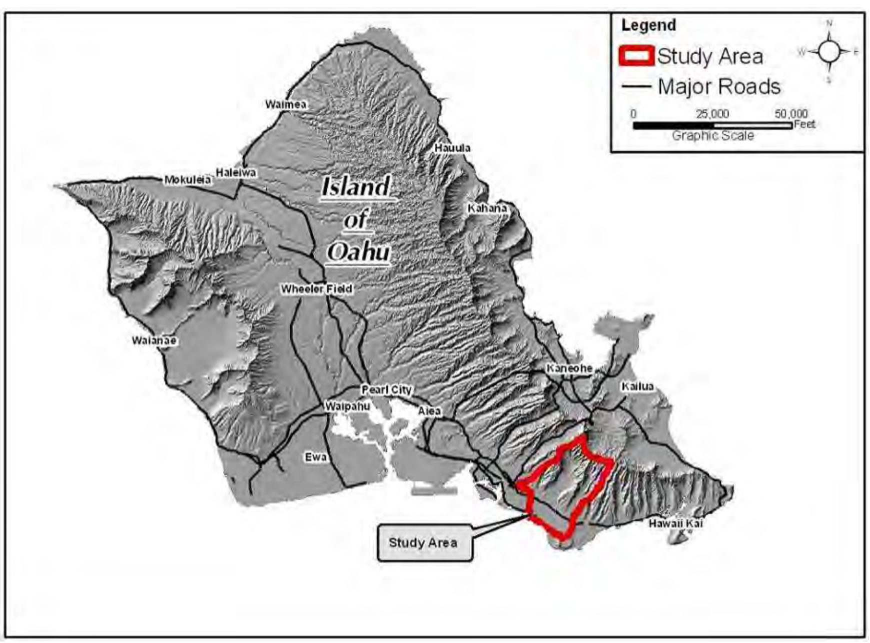

Figure 2-1. Ala Wai Watershed Location Map

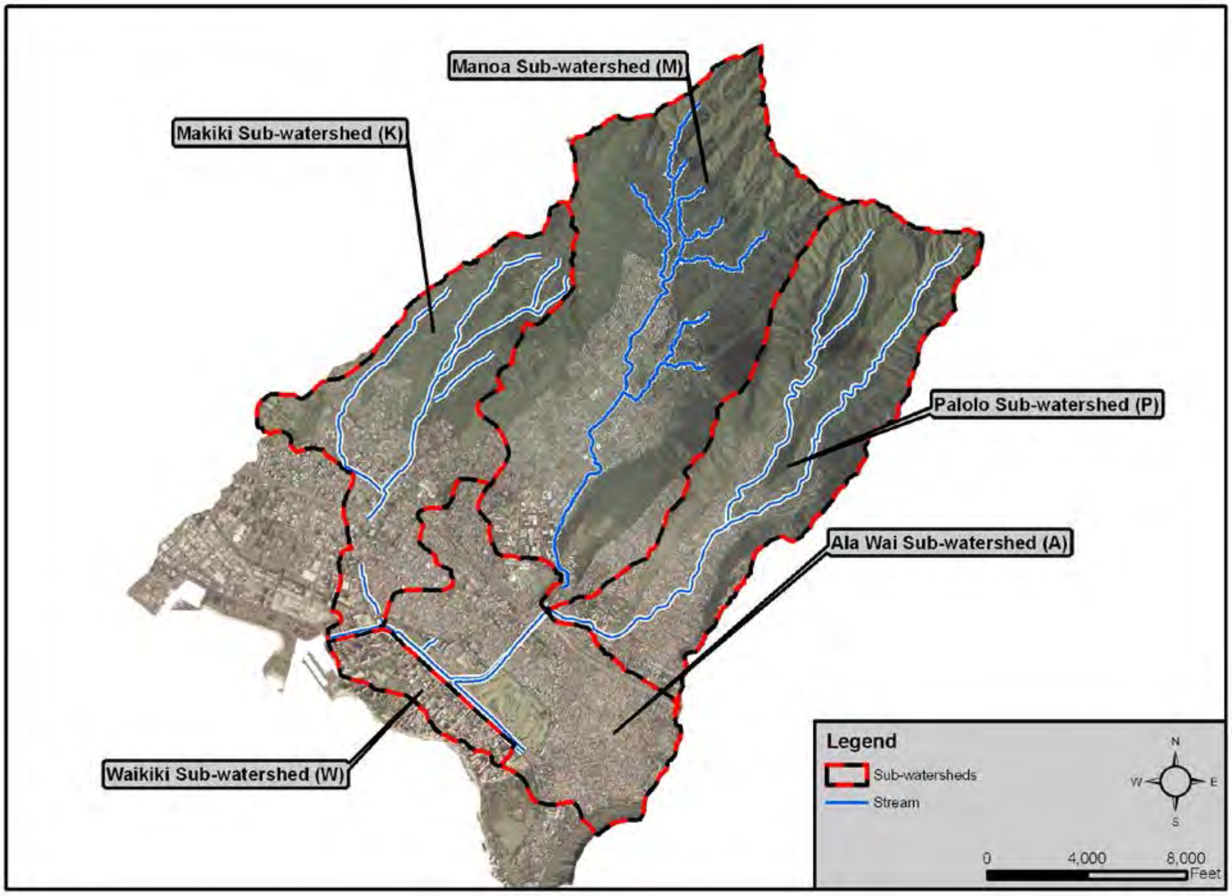

Figure 2-2. Major Streams and Sub-watersheds of Ala Wai Watershed

2.1.1 Climate and Flood Hydrology

Hawaiʻi's subtropical climate is governed by northeasterly trade winds that regulate weather patterns. The trade winds rise over the Koʻolau Mountain ridges, creating high moisture and orographic rainfall in the mountainous regions. These regions, such as the valley systems of Makiki and Mānoa typically receive more than 160 inches of annual rainfall, whereas the Pālolo valley system receives less annual rainfall (Giambelluca 1984). Generally, rainfall amount decreases as one moves down the valley systems to the southern coast of Oʻahu, and so the low-lying areas of the Ala Wai Canal and Waikīkī receive about 30 inches of annual rainfall. The wet winter season occurs from October to April, and the dry summer season occurs from May to September. It should be noted that the three severe storms described for this study occurred in October, December, and March, during the wet winter season. Temperatures on Oʻahu fluctuate according to the season, with the winter temperature averaging a high of 77 degrees Fahrenheit (°F) and a low of 64°F. In the summer, temperatures average a high of 81°F and a low of 70°F (NWS 2008).

Floods on Oahu, other than those generated by high ocean waves, are caused by high intensity rainfall. Most major rainstorms that bring flood-producing rainfall are caused by the non-trade wind or Kona wind conditions which occurred during the wet winter season. Rainstorms can bring intense local showers affecting a small area or can blanket the entire island with rain. High-intensity rainfall, small drainage-basin size, steep basin and stream slopes, and little channel storage, produce floods that are flashy (Wong, 1994). Most drainage basins have rapid response to rainfall characterized by steep triangular hydrographs. Time to peak is usually less than 1 hour and even for large intense storms, the rise and recession of the flood hydrograph usually occurs with 6 hours.

2.1.2 Geology and Soils

The valleys and gulches forming the Ala Wai Watershed are incised into the Koʻolau Volcano. The Koolau lavas are divided into the Koʻolau Basalt and the Honolulu Volcanics. Both of these formations play an important role in the Ala Wai Watershed. The Koʻolau Basalt primarily consists of Pliocene aged shield stage tholeiitic basalt. The Honolulu Volcanics are composed of Pleistocene aged alkalic basalt, basanite, and nephelinite (Lagenheim and Clague, 1987). Holocene and Pleistocene sedimentary caprock is found at the seaward end of the watershed.

The rocks of the Koʻolau Basalt can be divided into three groups, lava flows (aʻa and pahoehoe), pyroclastic deposits, and dikes. The lava flows of the Koʻolau basalt are usually thin bedded with an average thickness of about ten feet (Wentworth and MacDonald, 1953). These beds are composed of aʻa and pahoehoe flows and pyroclastic deposits. Aʻa contains a solid central core between two gravely clinker layers. Pahoehoe flows are usually characterized by a smooth ropy texture. Pyroclastic deposits originate from explosive volcanism. They are composed of friable sand-like ash and indurated tuff deposits. Dikes are thin near vertical sheets of rock that intruded or squeezed into existing lava flows or pyroclastic deposits.

The Honolulu Volcanics erupted much later than the Koʻolau Basalt and overlay the deeply eroded Koʻolau Volcano and its associated alluvial deposits. In Ala Wai they are composed of lava flows and ash and tuff. The lava flows have flow structures similar to the Koʻolau Basalt. The pyroclastic deposits are characterized by easily erodable, sand-like ash and relatively soft and easily erodable tuff. The Sugar Loaf flow which outcrops in cliffs in the UH Quarry poured down from Sugar Loaf on the northwest side of Mānoa Valley and pushed the lower section of Mānoa Stream to the southeast.

The caprock is composed of a wedge of terrestrial and marine sediments. It forms a coastal plain about 8000 feet wide in the Ala Wai area. The caprock is over 1000 feet thick in the seaward areas of the watershed (Wentworth, 1951). Near the ocean, much of the caprock has been covered with artificial fill.

Mānoa and Pālolo valleys are deeply eroded amphitheater shaped valleys that was later backfilled with alluvium and Honolulu Volcanic deposits. The original valleys were probably "V" shaped but the alluvial and volcanic fill material has formed a broad, flat-bottomed valley. The valley fill material is weathered at the surface but despite the heavy rainfall is probably fresh and unweathered in the subsurface. The ridges and valley walls of Mānoa and Pālolo Valleys are generally composed of Koʻolau Basalt (In some areas Honolulu pyroclastics drape the walls). The layered flows of Koʻolau Basalt have eroded into steep weathered cliffs which facilitate rapid runoff. Dikes in the back of the valleys impound groundwater at high elevations which contributes to perennial streamflow.

The altitude within the watershed ranges from mean sea level along the coastal areas, to 40 feet near the confluence of Mānoa and Pālolo Streams, and approximately 2,400 feet in the mountains. Several soil groups are found in the Ala Wai Watershed. The Lualualei-fill land-Ewa association is a well-drained soil that may be found in the lower elevations. These soils have fine textured or moderately fine-textured subsoil or underlying material. The upper watershed is comprised of rock land-stony steep land association. These soils are generally found on steep to precipitous lands and are well-drained to excessively drained (MacDonald et al. 1970).

2.2 Makiki Sub-Watershed

The Makiki sub-watershed is the westernmost of the Ala Wai Canal drainage sub-watersheds, and drains 1,850 acres or 2.89 square miles of land. Makiki Stream, which is approximately 3.5 miles long, drains the sub-watershed. The stream's tributaries include Kanahā Stream, the main tributary that connects to Makiki Stream via Kanahā Ditch (a long lateral channel of about 6,400 feet), Kānealole Stream, Moleka Stream, and Maunalaha Stream (Townscape 2003). The upper segment of the sub-watershed is in the Koʻolau Mountains and is bordered to the west by the Punchbowl Crater.

Whereas the upper sub-watershed is largely forested and undeveloped, the sub-watershed becomes more urbanized as one moves seaward. The upper Makiki sub-watershed has preservation land uses and is considered non-urbanized in this study. The lower Makiki sub-watershed includes the populated Makiki areas of Wilder Avenue, Mānoa Road, and McCully Street. The urbanized portion of the sub-watershed has residential and commercial land uses. Makiki Stream runoff from urban areas and minor streams ultimately discharges into the Ala Wai Canal between McCully Street and Kalākaua Avenue bridges.

2.3 Mānoa Sub-Watershed

Mānoa sub-watershed is located between the Makiki and Pālolo drainage sub-watersheds and drains 3,822 acres (5.97 square miles) of land from the Koʻolau Mountains to the confluence of Mānoa and Pālolo Streams. The upper sub-watershed has preservation land uses and is considered non-urban. In the upper sub-watershed area, several smaller tributaries feed into the Waihī and Waiakeakua Streams and flow into the Mānoa Stream. Mānoa Stream drains the sub-watershed. The Mānoa Stream passes by Noelani Elementary School, the University of Hawaiʻi at Mānoa (UHM) upper campus, and Kānewai Field, and finally meets the Pālolo Stream to form the Mānoa-Pālolo Canal.

Most of the ground surface in the upper sub-watershed is covered with primarily non-native forest, and the middle segment of the sub-watershed is highly urbanized. The natural path and the characteristics of the Mānoa Stream have been altered significantly. Urban culverts discharge storm runoff into the Mānoa Stream throughout the developed area.

2.4 Pālolo Sub-Watershed

The Pālolo drainage sub-watershed is the easternmost of the Ala Wai Canal drainage sub-watersheds, and drains 2,601 acres (4.07 square miles) of land. The Mānoa sub-watershed borders it to the west, and the Mauʻumae Ridge borders the sub-watershed to the east. The Pālolo sub-watershed drains the Koʻolau Mountains and extends down Pālolo Valley to Waiʻalae Avenue. For the purposes of this study, the upper Pālolo sub-watershed is considered non-urban because it has preservation land use. Pūkele Stream and Waiʻōmaʻo Stream are the sub-watershed's two tributary streams. These streams flow into the Pālolo Stream that drains mostly the urbanized portion of the sub-watershed. The land uses in this area are commercial and residential. The Pālolo Stream meets the Mānoa Stream as the Mānoa-Pālolo Canal. As the Pālolo Stream passes through the urban Pālolo area, the stream is a concrete-lined channel that was part of a flood control project constructed by the City and County of Honolulu.

2.5 Mānoa-Pālolo Canal Junctions

The Mānoa and Pālolo Streams meet as the Mānoa-Pālolo Canal downstream of Kānewai Field and immediately north of Waiʻalae Avenue. The Mānoa-Pālolo Canal discharges into the Ala Wai Canal downstream of the Ala Wai Golf Course. Even though Mānoa-Pālolo Canal drains a segment of the Ala Wai Canal sub-watershed, it does so through large storm drainage outfalls that empty directly into the canal. Thus, only junctions (not areas of the sub-watershed) of the Mānoa-Pālolo Canal were examined for this study, and the large outfalls that enter the canal drain 20,285 acres of land.

2.6 Ala Wai Canal Sub-Watershed

The Ala Wai Canal sub-watershed drainage system is 1805 acres (2.82 square miles) including the Mānoa-Pālolo Canal. Historically, the lower portion of Ala Wai Watershed consisted of wetlands and provided ample storage for heavy runoff from the watershed. Ala Wai Canal was designed to drain the wetlands formed by the streams and create dry land for Waikīkī resort development, and the canal was constructed in the 1920s. At the time of the Ala Wai Canal project, the urban development in the watershed was limited, but today the Waikīkī area is heavily urbanized. Runoff from Makiki, Mānoa, and Pālolo sub-watersheds contains suspended materials from the natural reaches of these watersheds, and, as a result, Ala Wai Canal has experienced significant sedimentation over the years.

For the purpose of this study, the Ala Wai Canal was modeled as a reservoir using USACE's HEC-HMS. Considering that the canal may be subject to backflow and meets the ocean at mean sea level, a reservoir model is appropriate due to the low elevation and likelihood of water storage. This assumption significantly affected the modeling of the Ala Wai Canal.

2.7 Waikīkī Sub-Watershed

The Waikīkī drainage sub-watershed is the southern-most and coastal area of the Ala Wai Canal drainage sub-watersheds, and drains 298 acres (0.47 square miles) of coastal land. The Waikīkī area is heavily urbanized and not only a vital center of the tourism industry on Oʻahu but also a popular residential, shopping, and nightlife area. Historically, the Waikīkī area was swamp land, and thus the sub-watershed is low-lying. The sub-watershed is characterized by impervious surfaces, and storm drainage runoff either flows as overland flow, flows directly into the ocean, or flows through the City drainage system directly into the Ala Wai Canal. The canal is at a similar elevation as the Waikīkī sub-watershed itself.

3 Data Gathered

The character of the land, the historical rainfall data, and historical stream flow data are relevant to the hydrological analysis of the Ala Wai Watershed. Data used for HEC-HMS model calibration included rain gage data, stream flow gage data, stage gage data, and tide gage data records of historical storms, and field surveys. These data were used to create rainfall intensity-duration-frequency curves. Rainfall data were the input for the HEC-HMS rainfall-runoff model calibration.

3.1 Rain Gages

Data sets from thirteen rain gages were used for the Ala Wai Watershed hydrologic analysis. Four of these rain gages are operated by the National Weather Service (NWS), four rain gages are operated by the BWS, three rain gages are operated by USGS, one rain gage is operated by the UHM, and one rain gage is privately operated. The characteristics of each gage are listed in Table 3-1. Figure 3-1 maps these rain gages in or nearby the study area, labeled by their name and identification number (ID). As shown, rain gages are located in a diversity of elevations and locations throughout the greater Ala Wai Watershed.

Typically, rainfall in upper elevations of the sub-watersheds is greater than that of the lower elevations. For the Makiki sub-watershed, the rain gage at the highest elevation is the Tantalus Peak gage at 1,665 feet above mean sea level (MSL). The Mānoa Tunnel rain gage at 650 feet above MSL is the highest for the Mānoa sub-watershed, and the Pālolo Tunnel rain gage is located at 995 feet above MSL. The lowest rain gage for the entire Ala Wai watershed is the Waikīkī Zoo gage at about 5 feet above MSL. It should be noted that three rain gages were located outside the study area. The Waikīkī Zoo rain gage (717.2) was used to represent the Ala Wai Canal and Waikīkī sub-watersheds. The Wihelmina Rise rain gage (721) was used to represent the middle Pālolo sub-watershed, and the Punchbowl Crater rain gage (709) was used to represent the lower Makiki sub-watershed. Figure 3-2 shows the annual rainfall distribution in the Ala Wai Watershed by major sub-watersheds.

Rain gage data sets vary according to whether records are taken in real time (typically 15-minute intervals) or daily. Records were used to extrapolate the rainfall hyetographs for all the sub-watersheds in the calibration basin models. Also, rain gage records provided essential data for three storms that were used to calibrate the HEC-HMS model. Those storms occurred on December 17–18, 1967; October 30, 2004; and March 31, 2006.

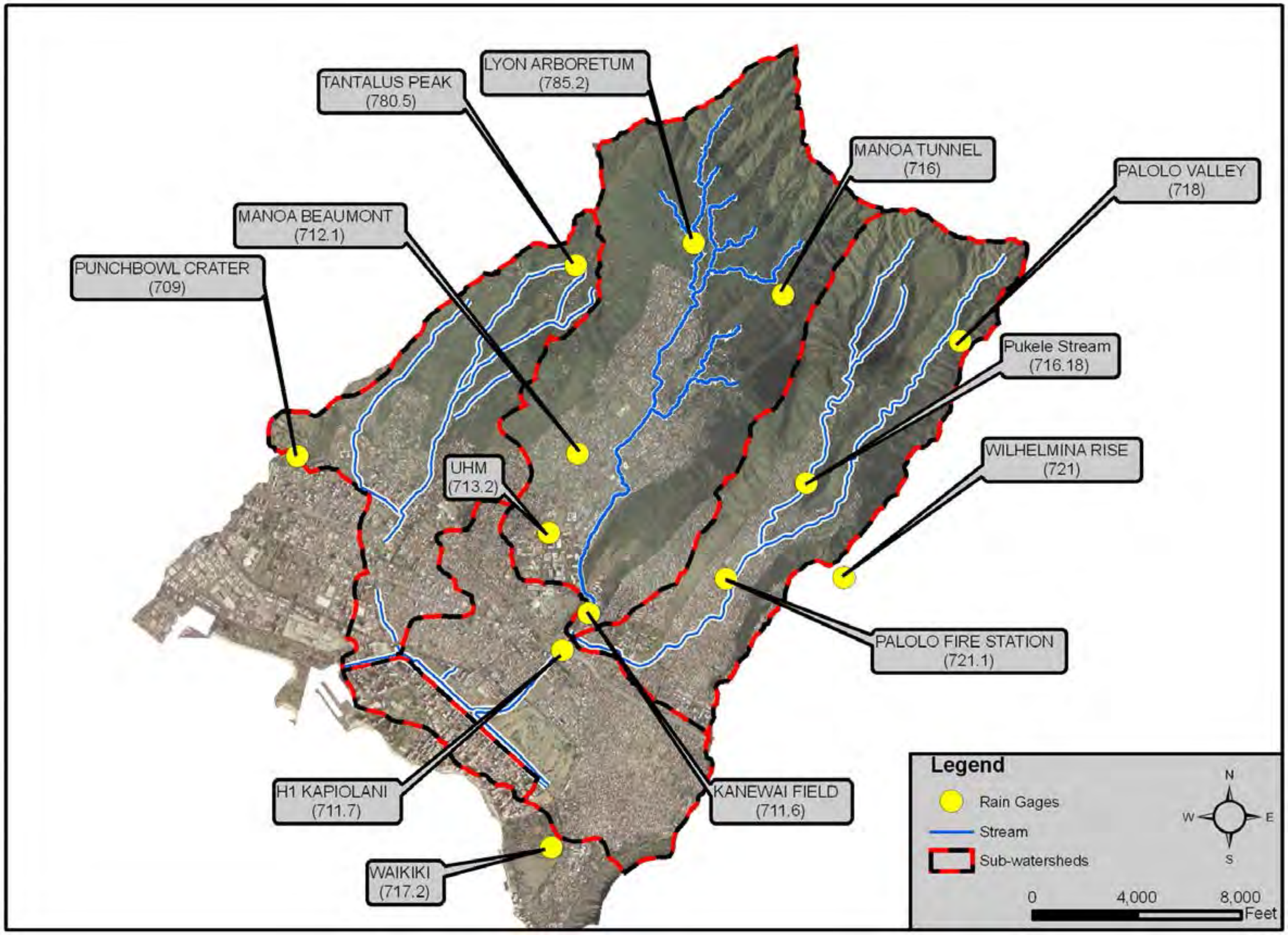

Figure 3-1. Ala Wai Watershed Rain Gages Used by Identification Number

Characteristics of Rain Gages Used

Name	ID	Latitude	Longitude	Elevation (ft)	Records	Real-time recording†	Daily recording*	Operator
Lyon Arboretum	785.2	21°20'08"	157°48'12"	500	1975– Present	√		NWS
Mānoa Tunnel	716	21°19'48"	157°47'36"	650	1927– Present		√	BWS
Kānewai Field	711.6	21°17'47"	157°48'56"	38	1999– Present	√		USGS
Mānoa Beaumont	712.1	21°18' 48"	157°49'00"	200	1947– Present		√	Private
UHM	713.2	21°18'18"	157°49'12"	120	1952– Present		√	UH
Pālolo Fire Stn.	721.1	21°18'00"	157°48'00"	190	1950– Present	√		NWS
Pālolo Tunnel	718	21°20'00"	157°49'00"	995	1926– Present	√		BWS
H-1 Kapiolani	711.7	21°17'22"	157°48'56"	20	2005– Present	√		USGS
Punchbowl Crater	709	21°18'48"	157°50'54"	355	1950– Present		√	NWS
Waikīkī Zoo	717.2	21°16'00"	157°49'00"	5	1957– Present	√		NWS
Wihelmina Rise	721	21°18' 00"	157°47'12"	1100	1927– Present		√	BWS
Pūkele Stream	716.18	21°18'36"	157°47'27"	345	1927– 2005	√		USGS
Tantalus Peak	780.5	21°20'00"	157°49'00"	1665	1927– Present		√	BWS

Table 3-1. Characteristics of Rain Gages Used.

† Real-time recording is by time intervals of 15 minutes. *Daily recording is 24-hour period

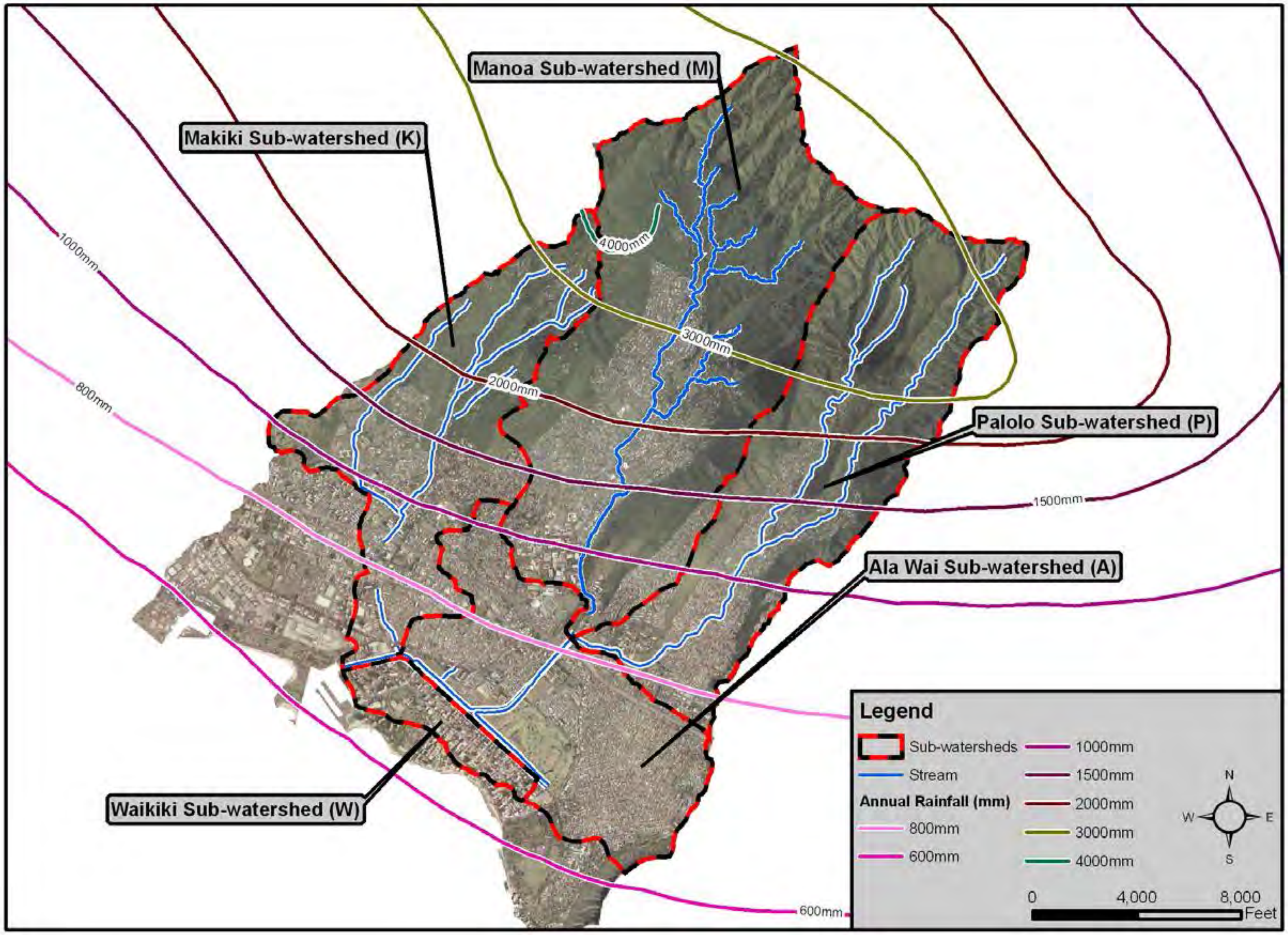

Figure 3-2. Annual Rainfall Distribution for Ala Wai Watershed by Major Sub-watershed

3.2 Stream Flow Gages

Historic stream gage records were used to develop the sub-basin analyses for the HEC-HMS model. Data sets came from nine stream gages throughout the Ala Wai Watershed, and these gages are shown in Figure 3-3 labeled with their USGS identification number. Stream gage data for three storms were essential for calibrating the HEC-HMS model (see calibration discussion in Section 3.8). These three storms occurred in 1967, 2004, and 2006 and are discussed in Section 3.8. Stream gage data for these events are limited depending on whether the gages' record continuously, such as by 15-minute intervals, or whether they simply record peak flow values. The characteristics of the stream gages are given in Table 3-2, and the stream flow gages are shown in Figure 3-3.

3.3 Stage Gages

The Waikīkī and Ala Wai Canal sub-watersheds are located on low-lying coastal land, and data from two stage gages were used in these areas, as shown in Figure 3-4. Stage gage data was essential for calibrating the Ala Wai Canal sub-watershed model detailed in Section 4.6. The nearest stage gage in the ocean was the National Oceanic and Atmospheric Agency's (NOAA's) tide level station 1612340 at Honolulu Harbor, which was used to calibrate the model. The other gage used was USGS 16247130 at Ala Wai Elementary School. These stage gages are located west of the study area as shown in Figure 3-2. Although there are no public published stage records, the local USGS office provided Oceanit with continuous stage data for the October 30, 2004, storm for calibration purposes (see Section 4).

Characteristics of Stream Gages Used									
Gage Location	**Waihī**	**Waiakeakua**	**Lowrey**	**Kānewai**	**Pūkele**	**Wai'ōma'o**	**Pālolo**	**Makiki**	**Mānoa-Pālolo**
Gage Number	16238500	16240500	16241500	16242500	1624400	16246000	16247000	16238000	16247100
Gage Location, Latitude	21°19'55"	21°19'52"	21°18'53"	21°17'47"	21°18'36"	21°18'34"	21°17'35"	21°17'02"	21°17'24"
Gage Location, Longitude	157°48'12"	157°48'08"	157°48'41"	157°48'56"	157°47'27"	157°47'11"	157°48'25"	157°50'22"	157°49'17"
Gage Elevation (ft)	289.84	294.5	294.5	38	344.78	373.66	95	10	5
Drainage Area (USGS, mi^2)	1.14	1.06	4.02	5.05	1.18	1.04	3.63	2.23	10.6
Drainage Area (mi^2)	1.19	1.07	4.22	5.643	1.146	1.036	3.62	2.49	10.34
Period of Continuous Record	1913–1983	1913–Present	---	1999–Present	1927–2004	1927–1971	1953–Present	---	1967–Present
Peak Flow Record Only	---	---	2003-2004	---	---	---	---	2003-2004	---
Number of Annual Peaks Available for Analysis	63	88	3	6	59	39	32	2	40

Table 3-2. Characteristics of Stream Gages Used

Characteristics of Stage Gages Used		
Gage Location	**Honolulu Harbor**	**Ala Wai Elementary School**
Gage Number	1612340	16247130
Gage Location, Latitude	21° 18.4'	21°17'16"
Gage Location, Longitude	157° 52.0'	157°49'51"
Gage Elevation (ft)	B.M. ELV. 8.06 Feet	5
Period of Continuous Record	1905-present	2003-2004

Table 3-3. Characteristics of Stage Gages Used

Note: B.M. ELV.= Bench Mark Elevation

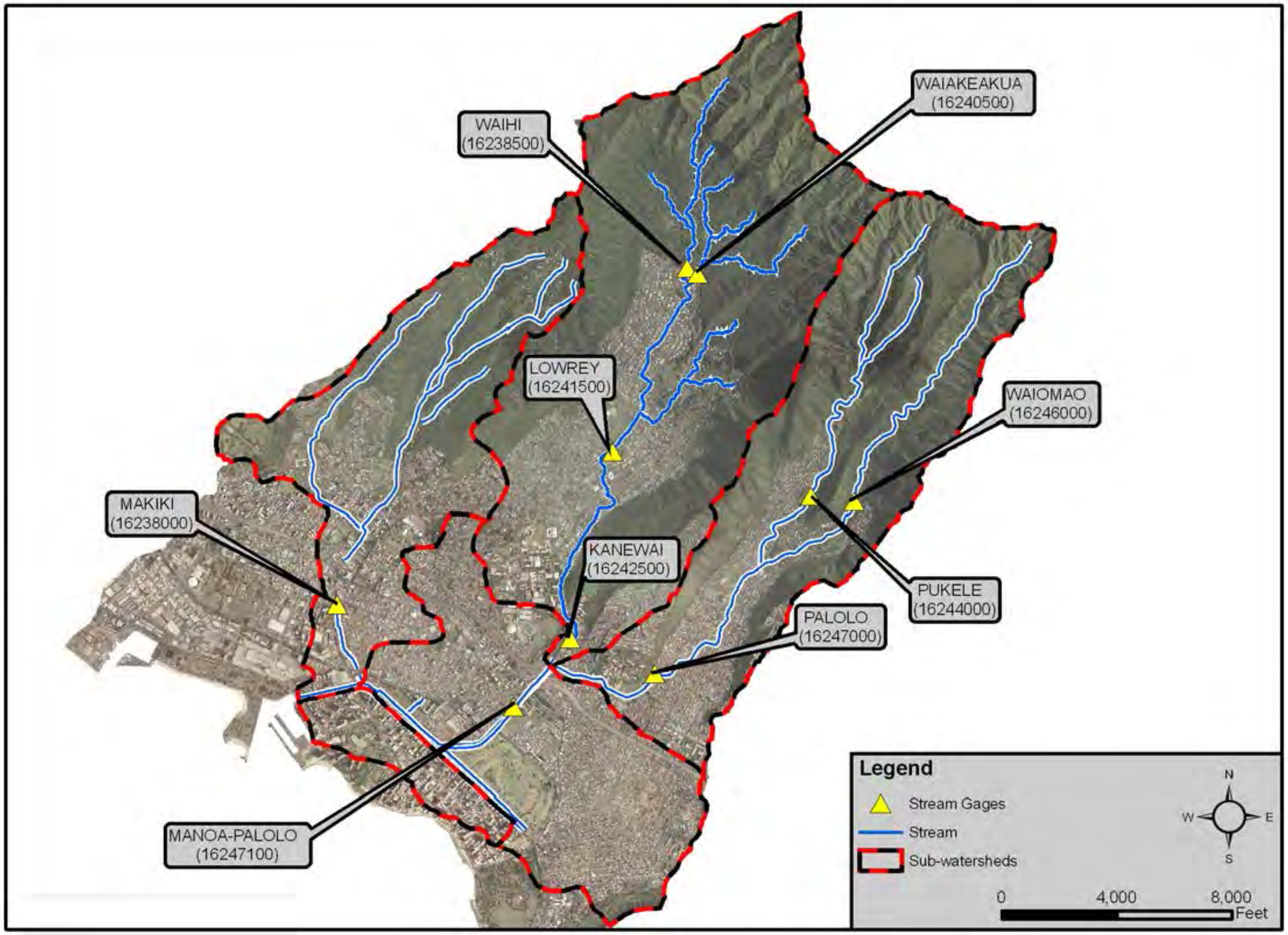

Figure 3-3. Ala Wai Watershed Stream Gages Used by ID Number

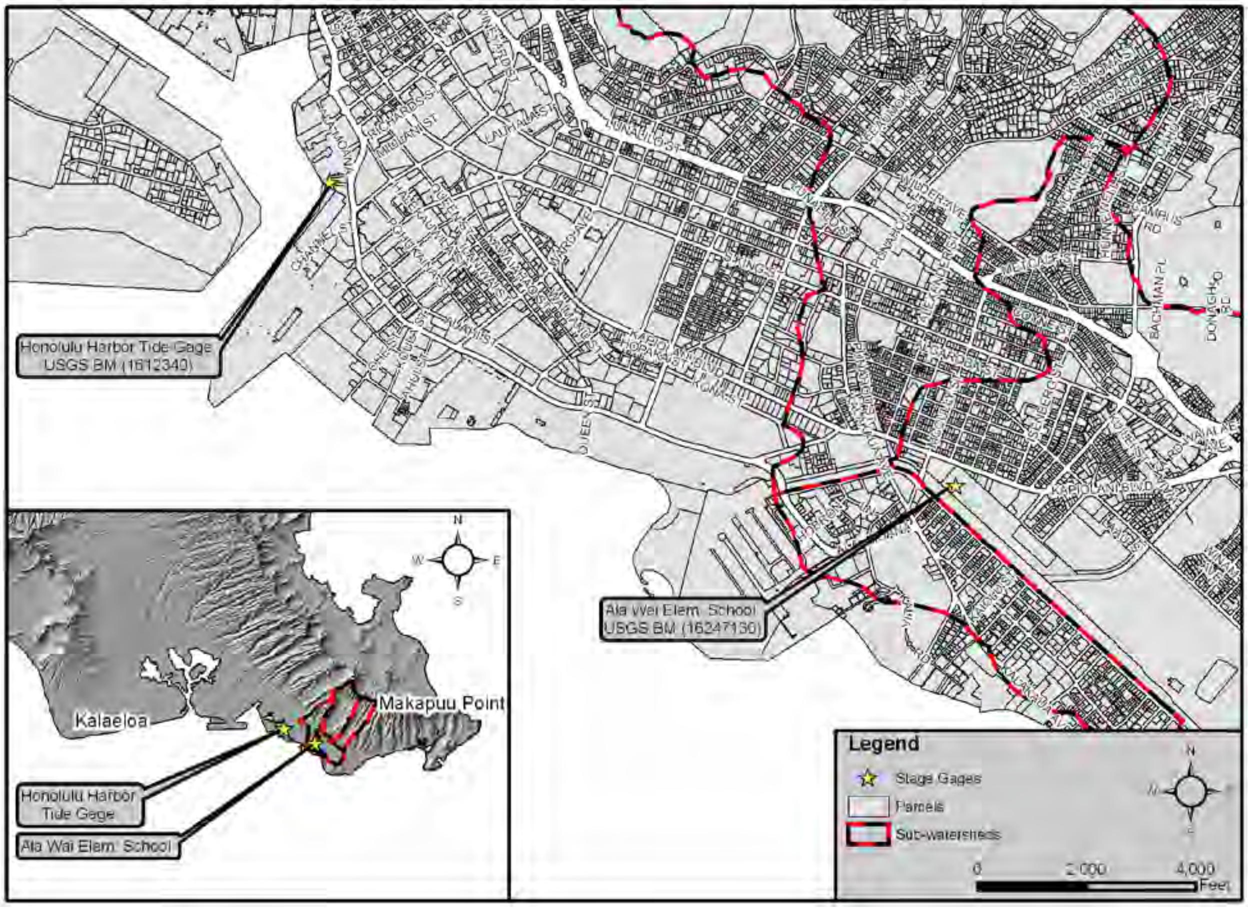

Figure 3-4. Ala Wai Watershed Stage Gages Used (with ID Number)

3.4 Drainage Systems

The City's municipal storm drainage system drains the sub-watersheds of the study area. Runoff from storms flows into the streams or drainage systems throughout the study area. The City's drainage maps were used to identify the locations of the existing storm drainage system. These maps provided information about the characteristics of drainage system segments, including whether the segments are natural or channelized and the size of outlets throughout the system. The drainage systems evaluation results were used in determining the sub-basins boundaries. For example, the boundaries of sub-basin K4 were mainly determined from drainage evaluation.

University of Hawai'i at Mānoa provided utility maps showing the drainage systems through the campus area. Existing conditions of the UHM's storm drainage system, such as the size of relevant culverts, were gathered from these maps. Detailed drainage systems information can be found in the *Final Drainage Evaluation Report Ala Wai Watershed Project* (Oceanit 2008). The drainage systems information within the UHM upper campus was used to determine the boundaries of sub-basin M12. Based on this information, the boundaries of sub-basins M12 were changed slightly. As a result, this sub-watershed's drainage area was different from the Manoa Watershed Study—it changed from 0.672 to 0.749 square miles.

3.5 Geospatial Data

Geospatial information [1] and field survey observations were used to determine hydrologic conditions, such as terrain roughness characteristics and stream channel cross sections. Information collected included LiDAR data and aerial maps. Numerous field visits to the various sub-watersheds of the study area were made over the course of January 2008 until September 2008 to confirm and/or describe any relevant existing condition of a drainage system facility or the existing conditions in a sub-basin.

LiDAR data were inputted into ArcView GIS 3.3 with the HEC-GeoHMS 1.1 extension to create a geospatial model of the Ala Wai Watershed. The HEC-GeoHMS (USACE 2003) model was used to delineate the initial sub-watershed boundaries, calculate sub-watershed areas, and determine flow path lengths and slopes. However, the sub-watersheds within the study area were not completely delineated by the HEC-GeoHMS model alone. The existing drainage infrastructure and the locations of potential conceptual design measures were important factors for sub-watershed delineation. The final sub-watershed delineation was the result of a combination of the HEC-GeoHMS model, an evaluation of the existing storm drainage system, and the potential locations of the conceptual design measures. LiDAR data were used to approximate the boundaries of sub-basins and sub-watersheds. In addition, ArcView GIS 3.3 and drainage maps were used to determine the boundaries of urbanized areas of the sub-watersheds' drainage areas because better resolution was available for evaluation.

3.6 Sub-Basin Delineation

For the purposes of this study, sub-watershed refers to the larger watershed areas of Makiki, Mānoa, Pālolo, and Waikīkī, and the term "sub-basin" refers to the smaller sub-watersheds within these sub-watersheds. These terms are used to avoid confusion. Also the term "sub-basin" is commonly accepted for the HEC-HMS model delineation of small drainage areas. Sub-basins provide clear boundaries for hydrologic study, and sub-basins were delineated according to a couple of assumptions. Sub-basin delineation assumes the following.

> The City's drainage systems can handle the storm runoff for all return periods from 2-year through 500-year storms.

This assumption takes into account all the storm runoff for storms, but not all storm runoff necessarily flows through storm drainage systems. According to the United States Department of Agriculture (1990; Module 206A), "Storm sewers generally handle only a small portion of a large

[1] The aerial images that were used for the hydrologic analysis are from the National Geospatial-Intelligence Agency (NGA) supplied by the USGS. The specifications for these images are 0.3 meter pixel size, rectified natural color image orthoimage. The working image was re-sampled to 1-meter pixel size.

The digital elevation LiDAR data used in this hydrologic analysis were obtained from AIRBORNE 1, with an accuracy of 4 elevation points per square meter. The original data were reprojected to North American Datum (NAD) 83 HARN 1993 Universal Transverse Mercator (UTM) Zone 4 meters. The grid size was 2 meters by 2 meters.

event. The rest of the peak flow travels by street, lawns, and so on to the outlet." This suggests that storm runoff flows along the natural geographic flow path and not necessarily through the storm drainage system. Based on the City's storm drainage standards, the drainage capacities with catchment areas greater than 100 acres should meet 100-year storm drainage standards; the drainage capacities with catchment areas equal to or less than 100 acres should meet 10-year storm drainage standards. Consequently, at junctions with contributing drainage systems, peak discharges may be lower than predicted. Similarly, at junctions where drainage system catchment areas are not considered, actual peak discharges may be higher than predicted.

Some delineations of sub-basins and assumptions about sub-basins were necessary for the low-lying areas of Mānoa-Pālolo Canal, Ala Wai Canal sub-watershed, and Waikīkī sub-watershed. Because Mānoa-Pālolo Canal receives drainage from other sub-watersheds with relatively large drainage systems, only the junctions in the Mānoa-Pālolo Canal were examined and there were no sub-basins delineated around the canal itself. Also, delineation for the Waikīkī sub-watershed was particularly problematic because some of its sub-basins drain directly into the ocean with a relatively small flow directed through the outfalls designated on the drainage maps.

It should be noted that all the hydrologic analysis results in this study for Mānoa sub-watershed were exactly the same as performed in the *Mānoa Watershed Project Final Hydrology Report* (Oceanit 2008) to keep consistency with the previous Mānoa Watershed Project hydrologic study. Another assumption was made about the UHM area in the Mānoa sub-watershed. The drainage area of sub-watershed M12 (UHM upper campus) was changed from the previous 0.672 square miles (Oceanit 2008) to 0.747 square miles. This drainage area determination accounts for the contribution of a 96-inch culvert storm drainage system at Dole Street Bridge. The characteristics of the storm sewer network were collected from the UHM Utility Map (2008).

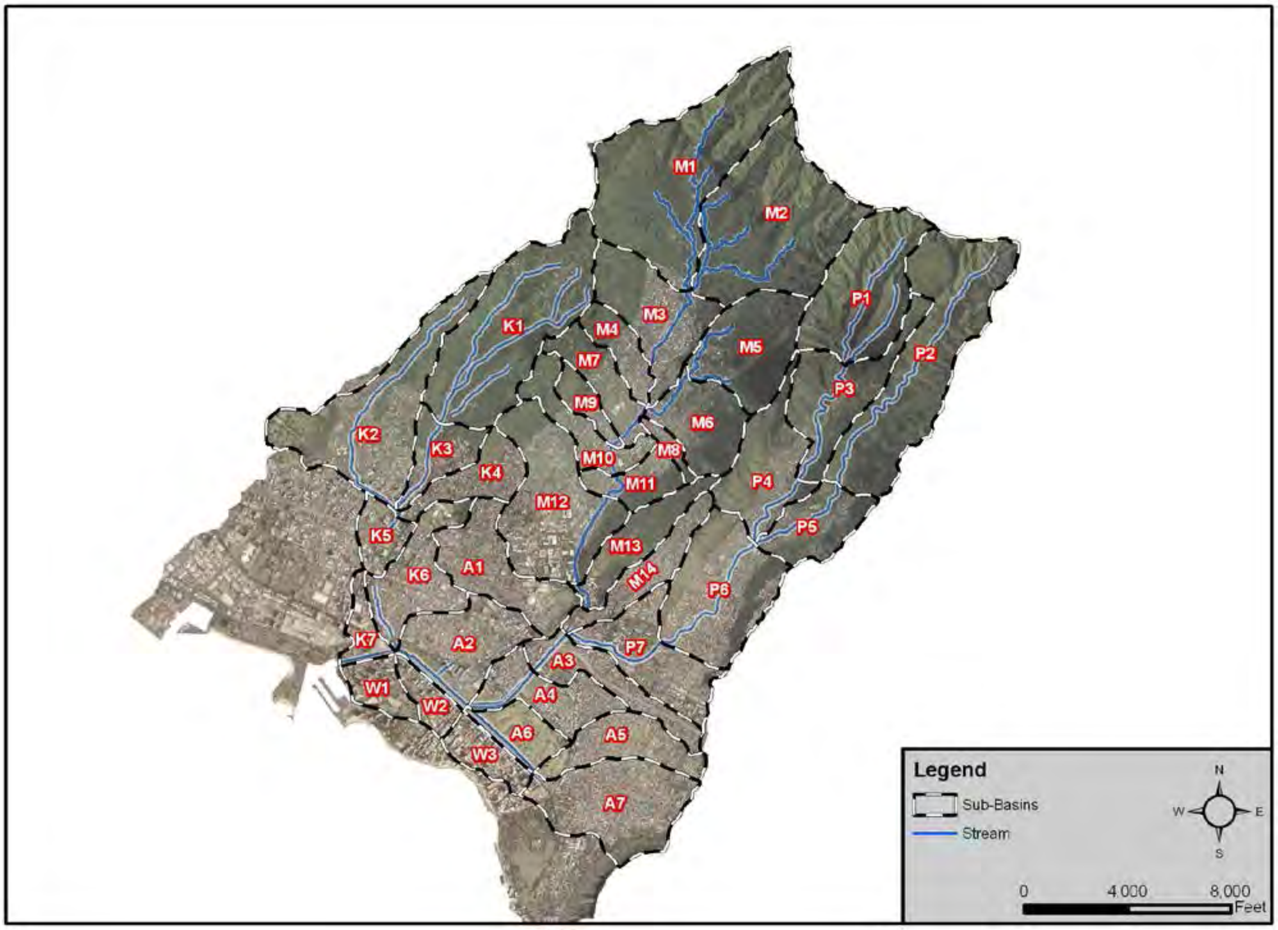

Figure 3-5. Ala Wai Watershed Sub-Basin Delineation

Ala Wai Watershed delineation of sub-basins was based on the junctions that are confluences of study area streams. The following table of sub-basin delineations designates the respective sub-watershed by the following.

- 'J' for junctions, or stream confluences, throughout the watershed
- 'K' for sub-basins in the Makiki sub-watershed
- 'M' for sub-basins in the Mānoa sub-watershed
- 'P' for sub-basins in the Pālolo sub-watershed
- Note that Mānoa-Pālolo Canal sub-watershed has junctions only and not sub-basins because other sub-basins empty into this canal but it does not drain its surrounding area
- 'A' for sub-basins in the Ala Wai sub-watershed; assumed to be a reservoir for the purposes of this study (see earlier discussion in Section 3.6)
- 'W' for sub-basins in the Waikīkī sub-watershed

Ala Wai Watershed Sub-Basin Delineation		
Sub-Basin/Junction	Sub-Basin or Junction Name	Drainage Area (mi^2)
MAKIKI		
KI	Upper Makiki Stream	1.00
K2	Kanahā Stream	0.85
K3	Middle Makiki Stream	0.22
K4	East Mānoa Road	0.25
JK1	Confluence of Makiki and Kanahā Streams	2.33
K5	Lower Makiki Stream	0.16
JK2	USGS Stream Gage near King St. 16238000	2.49
K6	Washington Middle School	0.40
JK3	Confluence of Makiki Stream and Ala Wai Canal	2.89
MĀNOA		
M1	Waihī	1.20
M2	Waiakeakua	1.07
JM1	Confluence of Waihī and Waiakeakua Streams	2.27
M3	Pawaina	0.51
M4	Poelua	0.18
M5	Woodlawn_Ditch 1	0.50
M6	Woodlawn_Ditch 2	0.35
JM2	Confluence of Mānoa Stream & Woodlawn Ditch	3.81
M7	Park	0.25
M8	Kahaloa	0.06
M9	Lowrey	0.11
JM3	Lowrey Ave. Bridge	4.22
M10	Woodlawn	0.26
JM4	Woodlawn Dr. Bridge	4.48
M11	Noelani	0.19
JM5	Mānoa Stream near Noelani Elementary School	4.67
M12	Dole (UHM campus)	0.75
JM6	Dole Street Bridge	5.42
M13	Kānewai	0.30
JM7	Kānewai Field Gage	5.72
M14	Saint Louis Heights	0.25
JM8	Just Upstream of the Confluence of Mānoa & Pālolo Streams	5.97

Table 3-4. Ala Wai Watershed Sub-Basin Delineation

Ala Wai Watershed Sub-Basin Delineation (Continued)		
Sub Basin or Junction Number	Sub Basin or Junction Name	Drainage area (mi^2)
PĀLOLO		
P1	Upper Pūkele Stream	0.67
P3	Middle Pūkele Stream	0.48
JP1	USGS Pūkele Gage 16244000	1.15
P2	Upper Waiʻōmaʻo Stream	1.04
P4	Lower Pūkele Stream	0.45
P5	Lower Waiʻōmaʻo Stream	0.31
JP2	Confluence of Pūkele and Waiʻōmaʻo Streams	2.94
P6	Pālolo Stream	0.68
JP3	USGS Pālolo Gage 16247000	3.62
P7	Waialae Avenue	0.45
JP4	Just Upstream of the Confluence of Mānoa & Pālolo Streams	4.07
MĀNOA-PĀLOLO		
JMP1	Confluence of Mānoa and Pālolo Streams	10.04
A3	H1 Freeway	0.30
JMP2	USGS Stream Gage 16247100	10.34
A4	Date Street	0.34
JMP3	Confluence of Mānoa-Pālolo and Ala Wai Canals	10.68
ALA WAI & WAIKĪKĪ		
A5	Kaimukī	0.32
A7	Diamond Head Drainage System	0.62
A6	Ala Wai Golf Course	0.20
W3	Kuhio	0.18
A1	UHM lower campus and Punahou School	0.45
A2	Mōʻiliʻili	0.47
W2	Kālakaua	0.13
A8	Hawaii Convention Center	0.12
W1	Ala Moana Blvd.	0.16
OUTLET	Mouth of Ala Wai Canal	16.21

Table 3-4 (Continued). Ala Wai Watershed Sub-Basin Delineation

Drainage systems collect the majority of runoff in Waikīkī, and thus, information about these systems was used to delineate the Waikīkī sub-watersheds. Most of the runoff flows through the City's drainage systems and discharges into the Ala Wai Canal. However, a small portion of runoff flows directly into the ocean. This small portion is overland flow or is emptied directly into the ocean by drainage pipes.

3.7 Storm Records Used for Calibration

Calibration of the HEC-HMS model relied on sub-basin analysis that used available records of three storms in December 17-18, 1967; October 30, 2004; and March 31, 2006. However, partial stream flow data were available for some gages and junctions had different recording equipment. Below is a list of the records available by location and storm. The locations refer to the HEC-HMS model layout.

- A partial data set from JM3 (Lowrey Ave. Bridge) from the 2004 storm was used for calibration
- At M2 (Waiakeakua sub-basin), peak flow data were used for the 1967 storm, and real-time data were used for the 2004 and 2006 storms
- At JMP2 (USGS stream gage 17247100 at Kaimukī High School), peak flow data were used for the 1967 storm, and real-time data were used for the 2004 and 2006 storms.
- At JP1 (USGS Pūkele Stream gage), peak flow data from the 1967 storm were used, and real-time data from the 2004 storm were used
- At JP3 (USGS stream gage 17247000 at Pālolo Stream), peak flow data from all three storms were used, but some of these data were discarded because they were clearly inaccurate—comparison to other gage readings downstream during the same storm showed clear inconsistencies

3.7.1 December 1967 Storm

On December 16, 1967, a surface weather front appeared to be stationary west of Hawaiʻi (DLNR 1968). Torrential rains started falling on Oʻahu around the middle of the night on December 17. Many rainfall stations reported excessive rainfall during the storm. Pālolo Valley, Waiʻalae-Kāhala, Niu Valley, and Waimānalo suffered extensive flood damage. Rainfall amounts registered in the windward area had a rainfall frequency of about a 25-year storm (DLNR, 1968). The Tantalus Peak rain gage registered 5 inches of rainfall for a 3-hour period ending at 3:00 AM. The Pālolo Tunnel rain gage, maintained by the BWS, recorded 10.06 inches between the middle of the night and 8:00 AM hours, with 2.4 inches from 4:00 AM to 5:00 AM. The rainfall intensity was almost uniformly distributed from the coastal area to the Koʻolau Mountains. The USGS stream gage 16247000 at the Pālolo Stream recorded a record high peak discharge of 4,270 cubic feet per second (cfs); the USGS stream gage 16247100 at the Mānoa-Pālolo Drainage Canal recorded its highest estimated discharge at 10,100 cfs.

3.7.2 October 2004 Storm

A storm on October 30, 2004, that caused flooding in the Mānoa Valley was characterized as about a 20-year storm (NWS 2005). This return period corresponds to a 5% probability of occurrence. The persistent and heavy rainfall created swift and high stream flows that were recorded throughout the Mānoa Stream by various rain and stream gages. The heaviest rainfall happened around 7:30 PM, at which time the Lyon Arboretum rain gage recorded 1.29 inches in 15 minutes. The gage records for the October 2004 storm were used to calibrate the HEC-HMS model.

3.7.3 March 2006 Storm

On March 31, 2006, a strong storm caused the NWS to issue flash flood warnings for O'ahu because rain fell on already saturated ground. The storm moved over the windward (eastern) half of O'ahu during the late morning, and rainfall of 1 to 2 inches were recorded within one-hour periods by several NWS gages (NWS 2006). The NWS Waimānalo rain gage recorded over 3 inches of rainfall within a two-hour period. During the six weeks prior to this storm, O'ahu had experienced heavy rains that saturated lands on the windward side of the island. The March 31 rainfall, coupled with the saturated character of the land, produced flash floods throughout the island (NWS 2006). The Moanalua, Makiki, and Mānoa Streams overtopped their banks, and residents of Mānoa valley were alerted of flash flooding in the area. Various intersections and flooding forced the partial closure of the area's major highway, H-1 Freeway, and downtown streets were clogged with traffic (Pacific Business News 2006).

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4 Hydrologic Analysis Procedure

Hydrologic analysis of sub-watersheds of the Ala Wai Watershed utilized up to five hydrologic modeling methods. Given the HEC-HMS model layout for the Ala Wai Watershed, the hydrologic analyses for sub-watersheds were completed on the basis of the existing conditions—particularly whether or not sub-watersheds are urbanized. For the sub-watersheds without much urbanized area, hydrologic models were calibrated using the storm records outlined in Section 3.8. The hydrologic model, as shown in Figure 4-1 was based on the sub-watersheds delineated. These sub-watersheds include the upper Makiki, upper Mānoa, and upper Pālolo. Thus, Sections 4.2 through 4.5 outline the necessary parameters that were calculated: rainfall amount, time of concentration, and curve numbers. As mentioned earlier, the Clark Unit Hydrograph was used as the transform method for these areas that are not urbanized.

For the sub-watersheds with more urbanized area, the hydrologic models used the Kinematic Wave Transform Method. Section 4.7 provides the Kinematic Wave Transform Method analyses of the urbanized areas of the Ala Wai Canal and Waikīkī sub-watersheds, alongside the Mānoa-Pālolo Canal junctions considered.

4.1 Hydrologic Model Layout

Stream junctions of interest that are listed in Table 3-3 are illustrated as the final hydrologic model layout as shown below in Figure 4-1.

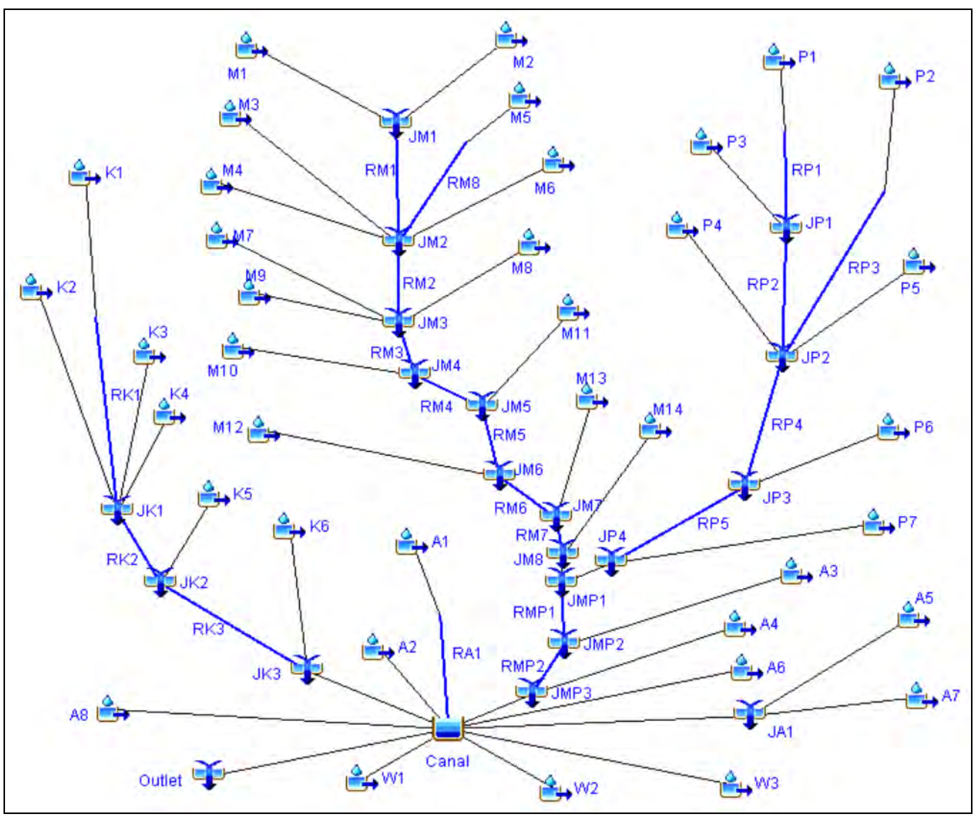

Figure 4-1. Ala Wai Watershed HEC-HMS Model Layout

4.2 Meteorological Model

The storm rainfall amounts that were the input for the hydrologic model are considered the meteorological model. The rainfall and stream flow data were collected from rain gage and stream flow gage records as available for the study area (see Sections 3.1 through 3.2).

4.2.1 Rainfall Amount Determination

Rainfall amount determination was necessary for 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent chance exceedance storms. These amounts were interpolated and/or extrapolated from "Rainfall Frequency Study for O'ahu", Report R-73, by Giambelluca, Lau, Fok and Schroeder (1984). For the 1-, 6-, and 24-hour rainfall amounts for the recurrence periods of 50, 10, 2, and 1 percent chance exceedance, values (shown in Table 4-1) were obtained directly from R-73 (Giambelluca 1984). The rainfall depths from R-73 were plotted, and the resulting smooth curve-function was used to estimate the rainfall depths that were not directly shown in R-73. Thus, for the percent chance exceedance storms less than the 1 percent storm, the rainfall amounts for various durations between 1 hour and 24 hours were determined from the duration nomographs presented in R-73. These curves are shown in Figure 4-3. The 0.5 and 0.2 percent chance exceedance storms' rainfall amounts were estimated by extrapolation using the rainfall depths relationships above the 1 percent chance exceedance storm. Rainfall values less then 1-hour were computed using 1-hour value. According to R-73, the 30-, 15-, and 5-minute rainfall values were determined by multiplying the 1-hour value by 0.714, 0.539, and 0.264, respectively.

Flow in the upper sub-watersheds may be underestimated due to sudden rainfall events that concentrate quickly as runoff because of high amounts of rainfall. Conversely, low rainfall is apparent in the lower sub-watersheds, and the relatively flat topography lends to underestimates of peak flows because runoff along the coastal areas may flow directly into the ocean. Thus, rainfall presented here is an average, based on the center point of the sub-basin and interpolated and extrapolated from the rainfall data available. The center point of each sub-basin was determined using the geospatial data discussed in Section 3.5. It should be noted that the 2001 Ala Wai Flood Study (USACE 2001) used a different approach for determining one rainfall value by averaging rainfall in the upper watershed and lower watershed rather than by averaging by the entire watershed.

Rainfall Intensity Duration Values for the Ala Wai Watershed

Percent Chance Exceedance	Recurrence Interval Year	Duration								
		5-min	15-min	30-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
50%	2	0.40	0.81	1.07	1.50	2.20	2.65	3.50	4.40	5.30
20%	5	0.49	1.00	1.32	1.85	2.80	3.40	4.45	5.70	7.15
10%	10	0.63	1.28	1.70	2.38	3.35	4.10	5.50	7.00	8.60
5%	20	0.70	1.43	1.89	2.65	3.80	4.65	6.25	8.05	10.05
2%	50	0.83	1.70	2.25	3.15	4.35	5.35	7.20	9.45	11.80
1%	100	0.91	1.86	2.46	3.45	4.85	6.00	8.25	10.90	13.65
0.5%	200	1.02	2.08	2.75	3.85	5.35	6.55	9.15	12.10	15.20
0.2%	500	1.16	2.37	3.14	4.40	6.10	7.55	10.40	13.65	17.00

Reference: Giambelluca et al. (1984), DLNR Report R-73

Table 4-1. Determined Rainfall Intensity Duration Values in inches for Ala Wai Watershed, Oahu, Hawaii. Note: rainfall intensity frequency data determined from maps and nomgraphs in Giambelluca et, 1984, DLNR Report R-73.

4.2.2 Rainfall Intensity-Duration-Frequency Curves

The rainfall-depth duration curves graph in Figure 4-2 shows the rainfall data as determined in average amounts for the percent chance exceedance storms. The rainfall amounts are for a 24-hour period, and were converted to intensity-duration-frequency (IDF) curves to offer rainfall intensities according to the range of storms examined (see Figure 4-3). The IDF curve is a crucial input into the HEC-HMS model analysis.

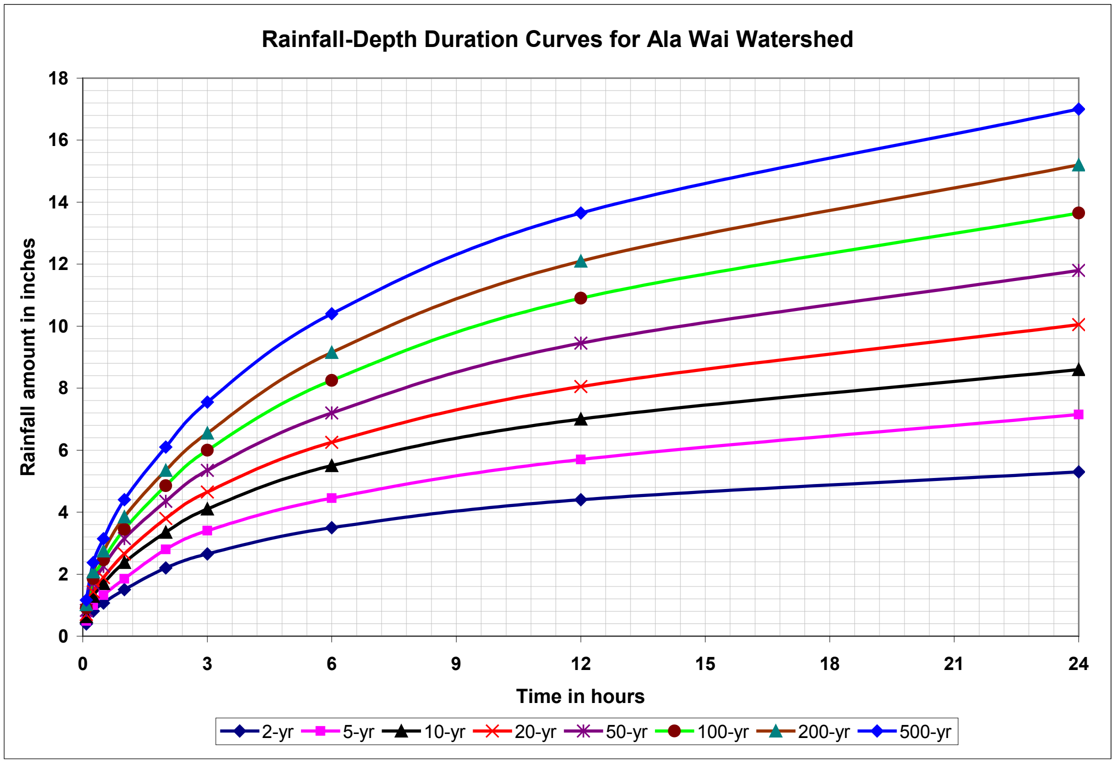

Figure 4-2. Rainfall-Depth Duration Curves for Ala Wai Watershed

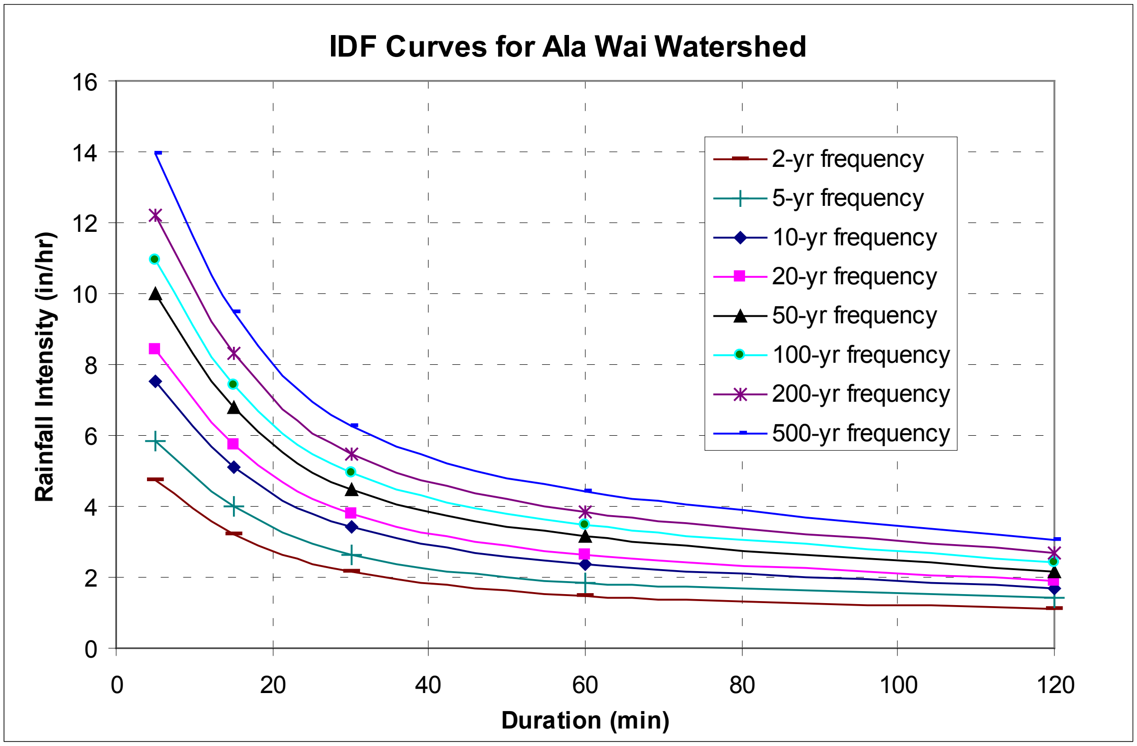

Figure 4-3. IDF curves for Ala Wai Watershed

4.2.3 Time of Concentration Calculation

The Clark Unit Hydrograph requires the parameter of the time of concentration (T_c) for each sub-basin. According to the TR-55 method, three types of flow path constitute the water flow: sheet flow, shallow concentrated flow, and channel flow; these three flows were added together to calculate time of concentration. According to the NRCS's Technical Report 55 (1986), "Time of concentration is the time for runoff to travel from the hydraulically most distant point of watershed to a point of interest within the watershed." The majority of the flow path may be channel flow as appropriate. Calculation of time of concentration is necessary for preparing the transform method for a unit hydrograph. The TR-55 velocity approach method was used to calculate time of concentration; that means the traveling time is a function of watercourse length and the velocity. The average velocity is a function of watercourse, slope, and type of channel.

A certain number of assumptions were made regarding sheet flow. The sheet flow segment describes the time period from raindrop impact until overland flow accumulates to a depth of about 0.1 foot, and one assumption made for time of concentration calculations was that the flow length for the stream reaches analyzed were not longer than 100 feet. The sheet flow segment T_c is calculated using Manning's kinematic solution, dependent on Manning's roughness coefficient *n*, the flow length, the rainfall amount, and the land slope. According to the SCS training material module 206A, "in most watersheds the overland [sheet] flow length is probably about 50 ft." (USDA, 1990) A maximum length of 100 feet is allowed in WinTR-55, and SCS suggests that a visit to the watershed is the best manner of determining the appropriate sheet flow length. Because this study lacked the appropriate observations for sheet flow during site visits, and considering previous studies and engineering judgement, a sheet flow length of 80 feet was set for all sub-watersheds in the Ala Wai Watershed for the calculation of time of concentration.

Overall, the flow length was determined from the City drainage maps and the known characteristics of the stream reach. Also, estimated flow length and land slope data were gathered from the geospatial data collected (see Section 3.5) using ArcView GIS 3.3. LiDAR topographic data and 5-foot elevation contours were used to calculate the slope of each sub-watershed.

4.2.4 Manning's *n* Roughness Coefficients

The surface Manning's roughness coefficients, based on the ground surface conditions, were determined as either 0.4 (woods with light underbrush) or 0.24 (dense grasses) using Table 3-1 from TR-55 (NRCS 1986). Where storm drainage systems are present in the sub-watershed, the appropriate flow path was used to estimate the time of concentration. Drainage pipe flow not under a pressure condition is treated as a portion of channel flow. The wetted perimeter condition assumes the full-flow condition for the drainage system pipes and the natural channel of the streambed. Altogether, the Manning's roughness coefficient for storm drainage facilities was selected as 0.015.

TR-55 Method Time of Concentration Parameters

Sheet Flow Characteristics					Shallow Concentrated Flow			Channel Flow Characteristics					Time of Concentration
Sub-Basin	**Manning's *n***	**Flow Length (ft)**	**Two-Year 24-hour Rainfall (in)**	**Land Slope**	**Surface Description**	**Flow Length (ft)**	**Slope**	**Flow Length (ft)**	**Cross-Section Area (ft^2)**	**Wetted Perimeter(ft)**	**Channel Slope**	**Manning's *n***	**T_C (hr)**
K1	0.4	80	5.3	0.450	Unpaved	1200	0.218	7200	30	19	0.174	0.035	**0.202**
K2	0.4	80	5.3	0.375	Unpaved	1150	0.278	9900	20	18	0.090	0.035	**0.311**
K3	0.24	80	5.3	0.313	Paved	1850	0.305	3100	30	19	0.042	0.035	**0.170**
K4	0.24	80	5.3	0.405	Paved	1450	0.365	5200	7.07	9.42	0.042	0.015	**0.165**
M14	0.24	50	5.3	0.250	Paved	1200	0.150	4150 1200	4.91 160	7.85 48	0.128 0.017	0.015 0.035	**0.152**
P1	0.4	80	5.3	0.260	Unpaved	1600	0.450	4850	40	24	0.159	0.040	**0.189**
P2	0.4	80	5.3	0.200	Unpaved	1850	0.172	9200	40	24	0.090	0.035	**0.313**
P3	0.24	80	5.3	0.306	Unpaved	2300	0.321	5500	48	20	0.061	0.035	**0.203**
P4	0.24	80	5.3	0.280	Unpaved	2800	0.285	1950 2400	3.14 48	6.28 20	0.115 0.0375	0.015 0.035	**0.215**
P5	0.24	80	5.3	0.260	Paved	800	0.285	700 4050	1.77 48	4.7 20	0.236 0.0395	0.015 0.035	**0.163**
P6	0.24	80	5.3	0.270	Paved	1150	0.550	800 5600	4.9 120	7.85 48	0.0625 0.0187	0.015 0.018	**0.168**
P7	0.24	80	5.3	0.180	Paved	700	0.040	3100 3500	4.9 160	7.85 48	0.03 0.02	0.015 0.018	**0.218**

Table 4-2. TR-55 Method Time of Concentration Parameters

Table 4-2 shows the values for sheet flow, shallow concentrated flow, and channel flow that were used to calculate the times of concentration. The times of concentration range from 0.152 hours in the Mānoa 14 sub-basin to 0.313 hours in the Pālolo 2 sub-basin, as shown in Table 4-3.

4.3 Curve Numbers Calculation

Runoff curve numbers, according to the TR-55 method (NRCS 1986), were used to determine the loss method of the HEC-HMS. Soil types in the study area were identified, and assigned to their appropriate hydrologic soil group (HSG in Table 4-3) classification. Geospatial data collected were used to determine land cover appropriate to each sub-basin, and for the various sub-watersheds. The different types of land cover and associated curve numbers are shown in Table 4-3. For the specific sub-basins, curve numbers were multiplied by the areas of the soil types by sub-watershed. For each sub-watershed, the product of these calculations was averaged over the total sub-watershed area to arrive at a composite curve number.

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Calculation of Composite Curve Numbers for Ala Wai Watershed

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	LAND USE	A	B	C	Total	A	B	C	A	B	C	CN
A1	Bare Land	0.0	3.1	1.0	5.0	72	82	87	0.0	253.0	89.8	
	Evergreen Forest	0.5	0.2	0.2	0.8	30	55	70	13.6	11.2	11.2	
	Grassland	3.2	4.7	4.4	12.3	39	61	74	126.4	287.4	323.2	
	High Intensity Developed	13.9	73.8	34.6	122.3	89	92	94	1240.5	6791.8	3249.2	
	Low Intensity Developed	62.6	55.4	22.6	140.7	77	85	90	4821.2	4710.6	2036.1	
	Scrub/Shrub	2.1	3.2	3.4	8.7	30	48	65	63.4	152.5	222.4	
A1 Total		83.3	140.4	66.2	289.9	A1 Composite CN						84
A2	Bare Land		1.6	6.1	7.7	72	82	87	0.0	132.6	527.6	
	Cultivated Land			0.2	0.2	77	86	91	0.0	0.0	20.2	
	Evergreen Forest		0.1	0.4	0.6	30	55	70	0.0	6.9	30.8	
	Grassland		4.1	12.5	16.6	39	61	74	0.0	250.9	926.5	
	High Intensity Developed		106.5	109.9	216.4	89	92	94	0.0	9799.5	10327.7	
	Low Intensity Developed		13.0	19.8	32.8	77	85	90	0.0	1105.8	1781.7	
	Scrub/Shrub		2.1	7.2	9.2	30	48	65	0.0	99.5	465.2	
	Water		0.1	15.1	15.1	98	98	98	0.0	8.0	1476.0	
A2 Total			127.5	171.1	298.7	A2 Composite CN						90
A3	Bare Land		1.9	0.0	1.9	72	82	87	0.0	157.9	0.0	
	Evergreen Forest		0.2	0.0	0.2	30	55	70	0.0	12.2	0.0	
	Grassland		5.6	0.0	5.6	39	61	74	0.0	342.0	0.0	
	High Intensity Developed		143.8	0.0	143.8	89	92	94	0.0	13229.4	0.0	
	Low Intensity Developed		38.5	0.0	38.5	77	85	90	0.0	3276.0	0.0	
	Scrub/Shrub		3.8	0.0	3.8	30	48	65	0.0	183.9	0.0	
A3 Total			193.9	0.0	193.9	A3 Composite CN						89
A4	Bare Land		3.0	0.9	3.9	72	82	87	0.0	244.7	79.4	
	Evergreen Forest		0.2	0.1	0.2	30	55	70	0.0	8.8	4.4	
	Grassland		23.6	6.0	29.6	39	61	74	0.0	1440.7	441.8	
	High Intensity Developed		122.7	5.0	127.8	89	92	94	0.0	11290.6	472.8	
	Low Intensity Developed		29.1	7.6	36.8	77	85	90	0.0	2477.6	685.8	
	Scrub/Shrub		7.9	6.5	14.3	30	48	65	0.0	378.5	419.3	
	Water		0.2	3.6	3.8	98	98	98	0.0	18.0	349.9	
A4 Total			186.7	29.6	216.3	A4 Composite CN						85
A5	Bare Land		1.2	0.2	1.4	72	82	87	0.0	98.4	19.3	
	Evergreen Forest		0.2	0.0	0.2	30	55	70	0.0	12.2	0.0	
	Grassland		12.2	2.2	14.5	39	61	74	0.0	745.9	165.1	
	High Intensity Developed		131.9	0.8	132.7	89	92	94	0.0	12138.5	72.6	
	Low Intensity Developed		48.7	4.7	53.4	77	85	90	0.0	4140.4	422.9	
	Scrub/Shrub		0.9	0.6	1.5	30	48	65	0.0	43.8	38.8	
	Water			0.4	0.4	98	98	98	0.0	0.0	38.4	
A5 Total			195.2	8.9	204.1	A5 Composite CN						88
A6	Bare Land		2.7	5.0	7.7	72	82	87	0.0	223.0	437.4	
	Evergreen Forest			0.5	0.5	30	55	70	0.0	0.0	32.1	
	Grassland		52.8	18.6	71.3	39	61	74	0.0	3219.5	1372.7	
	High Intensity Developed			5.1	5.1	89	92	94	0.0	0.0	478.9	
	Low Intensity Developed		0.1	3.1	3.1	77	85	90	0.0	5.9	275.9	
	Scrub/Shrub		4.5	15.0	19.5	30	48	65	0.0	215.9	977.5	
	Water		1.2	17.5	18.7	98	98	98	0.0	116.1	1714.1	
A6 Total			61.2	64.7	126.0	A6 Composite CN						72
A7	Bare Land		1.5	2.5	3.9	72	82	87	0.0	120.8	214.5	
	Evergreen Forest		2.0	0.1	2.1	30	55	70	0.0	111.6	4.2	
	Grassland	0.2	12.8	1.1	14.1	39	61	74	7.7	783.5	79.1	
	High Intensity Developed	0.0	238.4	3.8	242.3	89	92	94	3.5	21935.1	361.6	
	Low Intensity Developed	0.3	67.2	10.3	77.9	77	85	90	25.4	5715.0	928.2	
	Scrub/Shrub	0.8	15.5	38.1	54.4	30	48	65	24.3	744.6	2478.1	
	Water		0.0	2.2	2.2	98	98	98	0.0	0.3	211.5	
A7 Total		1.4	337.5	58.0	396.9	A7 Composite CN						85
A8	Bare Land			0.1	0.1	72	82	87	0.0	0.0	12.4	
	Grassland		0.1	0.4	0.6	39	61	74	0.0	8.7	32.9	
	High Intensity Developed		28.8	37.8	66.6	89	92	94	0.0	2647.5	3556.8	
	Low Intensity Developed		1.4	3.0	4.4	77	85	90	0.0	120.1	271.4	
	Scrub/Shrub		0.7	2.4	3.1	30	48	65	0.0	32.0	159.0	
	Water			4.3	4.3	98	98	98	0.0	0.0	424.2	
A8 Total			31.0	48.2	79.2	A8 Composite CN						92
K1	Evergreen Forest	268.5	1.4	66.9	336.8	30	55	70	8056.1	75.6	4681.3	
	Grassland	3.3		1.4	4.8	39	61	74	129.7	0.0	106.4	
	Low Intensity Developed	30.9	0.0	3.9	34.9	77	85	90	2382.0	0.7	354.0	
	Scrub/Shrub	216.6		48.9	265.5	30	48	65	6499.0	0.0	3178.2	
K1 Total		519.4	1.4	121.1	642.0	K1 Composite CN						40
K2	Bare Land	0.7		0.4	1.1	72	82	87	48.0	0.0	38.7	
	Evergreen Forest	97.2		51.4	148.6	30	55	70	2917.0	0.0	3595.5	
	Grassland	69.3	0.5	19.5	89.3	39	61	74	2702.9	27.6	1443.9	
	High Intensity Developed	34.4	29.8	14.5	78.7	89	92	94	3062.2	2742.4	1363.0	
	Low Intensity Developed	103.2	10.2	43.3	156.7	77	85	90	7949.2	863.6	3901.4	
	Scrub/Shrub	47.3	0.0	22.5	69.8	30	48	65	1419.1	1.1	1461.9	
K2 Total		352.2	40.4	151.7	544.3	K2 Composite CN						62

Table 4-3. Calculation of Composite Curve Numbers for Ala Wai Watershed

Calculation of Composite Curve Numbers for Ala Wai Watershed (Continued)

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	**LAND USE**	**A**	**B**	**C**	**Total**	**A**	**B**	**C**	**A**	**B**	**C**	**CN**
K3	Evergreen Forest	17.1	7.3	4.4	28.9	30	55	70	513.0	404.0	309.3	
	Grassland	5.7	2.2	0.7	8.6	39	61	74	221.6	131.5	52.9	
	High Intensity Developed	2.3	11.1	0.0	13.4	89	92	94	203.6	1024.5	0.0	
	Low Intensity Developed	55.4	18.0	2.9	76.3	77	85	90	4267.4	1533.9	258.1	
	Scrub/Shrub	7.7	1.2	7.1	16.0	30	48	65	230.3	58.4	462.2	
K3 Total		88.2	39.9	15.1	143.2	K3 Composite CN						**68**
K4	Bare Land	1.1		0.0	1.1	72	82	87	77.3	0.0	3.3	
	Evergreen Forest	3.1	0.3	0.0	3.4	30	55	70	93.4	15.9	0.0	
	Grassland	13.7	1.0	0.3	15.0	39	61	74	532.7	61.0	25.7	
	High Intensity Developed	4.8	6.2	0.0	11.0	89	92	94	426.9	573.0	2.0	
	Low Intensity Developed	100.1	9.9	0.0	110.0	77	85	90	7708.3	837.6	0.0	
	Scrub/Shrub	18.9	1.1	0.0	20.0	30	48	65	566.2	52.9	0.0	
K4 Total		141.6	18.5	0.4	160.5	K4 Composite CN						**68**
K5	Bare Land		0.4	0.0	0.4	72	82	87	0.0	36.5	0.0	
	Evergreen Forest		0.4	0.0	0.4	30	55	70	0.0	20.8	0.0	
	Grassland		4.4	0.0	4.4	39	61	74	0.0	266.0	0.0	
	High Intensity Developed		73.3	0.0	73.3	89	92	94	0.0	6748.0	0.0	
	Low Intensity Developed		21.6	0.0	21.6	77	85	90	0.0	1839.8	0.0	
	Scrub/Shrub		3.2	0.0	3.2	30	48	65	0.0	154.1	0.0	
K5 Total			103.4	0.0	103.4	K5 Composite CN						**88**
K6	Bare Land	0.3	0.7	0.2	1.2	72	82	87	21.0	54.7	19.3	
	Evergreen Forest	2.6		0.0	2.6	30	55	70	77.6	0.0	0.0	
	Grassland	0.3	7.6	3.2	11.2	39	61	74	12.2	466.6	237.3	
	High Intensity Developed	3.1	141.2	53.6	197.9	89	92	94	271.8	12991.1	5041.6	
	Low Intensity Developed	7.6	22.3	3.9	33.8	77	85	90	581.9	1898.8	349.7	
	Scrub/Shrub	4.6	5.8	0.3	10.6	30	48	65	137.4	276.9	18.8	
	Water			0.0	0.0	98	98	98	0.0	0.0	1.4	
K6 Total		18.4	177.6	61.3	257.3	K6 Composite CN						**87**
M1	Evergreen Forest	124.6	48.4	80.4	253.4	30	55	70	3738.1	2661.3	5628.2	
	Grassland	3.2	6.2	4.2	13.7	39	61	74	125.3	379.7	313.8	
	High Intensity Developed		0.7	0.9	1.5	89	92	94	0.0	63.5	80.5	
	Low Intensity Developed	0.3	7.6	5.4	13.4	77	85	90	26.2	643.0	490.4	
	Scrub/Shrub	51.6	40.2	393.3	485.1	30	48	65	1547.7	1929.2	25565.2	
M1 Total		179.7	103.1	484.3	767.1	M1 Composite CN						**56**
M10	Bare Land	0.6		1.1	1.7	72	82	87	44.8	0.0	96.3	
	Evergreen Forest	19.6		0.5	20.1	30	55	70	588.4	0.0	32.0	
	Grassland	5.2	0.1	3.0	8.3	39	61	74	201.0	7.9	222.3	
	High Intensity Developed	4.5	3.5	17.5	25.5	89	92	94	400.8	323.4	1641.2	
	Low Intensity Developed	40.3	4.3	33.4	77.9	77	85	90	3103.4	362.7	3003.1	
	Scrub/Shrub	24.9		9.2	34.1	30	48	65	746.9	0.0	600.2	
M10 Total		95.1	7.9	64.6	167.6	M10 Composite CN						**68**
M11	Bare Land			0.4	0.4	72	82	87	0.0	0.0	32.6	
	Evergreen Forest		1.0	11.4	12.3	30	55	70	0.0	52.6	795.4	
	Grassland	0.0	4.8	0.8	5.6	39	61	74	0.6	292.0	56.8	
	High Intensity Developed		1.2	4.7	5.9	89	92	94	0.0	108.6	445.2	
	Low Intensity Developed	5.1	17.4	23.8	46.2	77	85	90	389.1	1480.0	2139.9	
	Scrub/Shrub		1.2	49.9	51.1	30	48	65	0.0	57.4	3246.6	
M11 Total		5.1	25.5	91.0	121.6	M11 Composite CN						**75**
M12	Bare Land	2.9	0.5	0.6	4.0	72	82	87	208.8	37.5	54.7	
	Evergreen Forest	12.9	2.3	4.9	20.1	30	55	70	387.8	125.0	344.2	
	Grassland	20.5	9.6	6.8	36.9	39	61	74	799.3	588.6	500.5	
	High Intensity Developed	12.9	59.2	5.0	77.1	89	92	94	1150.7	5446.9	469.0	
	Low Intensity Developed	151.6	61.9	9.5	222.9	77	85	90	11674.1	5257.5	851.8	
	Scrub/Shrub	49.0	15.9	53.1	118.0	30	48	65	1471.2	761.7	3452.9	
M12 Total		249.9	149.3	79.9	479.1	M12 Composite CN						**70**
M13	Bare Land		1.0	0.2	1.2	72	82	87	0.0	78.0	19.4	
	Evergreen Forest		31.1	46.5	77.6	30	55	70	0.0	1712.6	3254.3	
	Grassland		1.0	3.1	4.0	39	61	74	0.0	60.4	226.4	
	High Intensity Developed		7.4	3.1	10.5	89	92	94	0.0	684.6	288.0	
	Low Intensity Developed		14.4	11.7	26.2	77	85	90	0.0	1226.4	1056.9	
	Scrub/Shrub		17.9	51.3	69.2	30	48	65	0.0	858.6	3334.6	
M13 Total			72.8	115.9	188.7	M13 Composite CN						**68**

Table 4-3 (Continued). Calculation of Composite Curve Numbers for Ala Wai Watershed

Calculation of Composite Curve Numbers for Ala Wai Watershed (Continued)

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	LAND USE	A	B	C	Total	A	B	C	A	B	C	CN
M14	Bare Land		1.2	0.7	1.9	72	82	87	0.0	99.5	63.2	
	Evergreen Forest		0.3	0.2	0.4	30	55	70	0.0	14.0	13.3	
	Grassland		6.3	1.7	8.1	39	61	74	0.0	387.3	126.8	
	High Intensity Developed		47.7	4.1	51.8	89	92	94	0.0	4389.8	387.0	
	Low Intensity Developed		76.8	12.4	89.2	77	85	90	0.0	6526.7	1118.4	
	Scrub/Shrub		7.6	3.5	11.2	30	48	65	0.0	366.4	229.7	
M14 Total			139.9	22.7	162.7	M14 Composite CN						84
M2	Evergreen Forest		92.1	91.6	183.7	30	55	70	0.0	5063.4	6414.8	
	Grassland		0.4	12.3	12.7	39	61	74	0.0	26.3	910.0	
	Low Intensity Developed		0.2	2.5	2.7	77	85	90	0.0	15.3	227.4	
	Scrub/Shrub		29.0	458.6	487.6	30	48	65	0.0	1392.0	29806.5	
M2 Total			121.7	565.0	686.7	M2 Composite CN						64
M3	Bare Land		0.2	0.0	0.2	72	82	87	0.0	18.2	0.0	
	Evergreen Forest	6.6	14.7	12.0	33.4	30	55	70	199.3	810.7	841.2	
	Grassland	7.9	20.1	3.3	31.3	39	61	74	309.9	1227.1	242.7	
	High Intensity Developed	13.4	20.7	5.2	39.3	89	92	94	1191.5	1907.7	488.2	
	Low Intensity Developed	20.5	54.6	14.0	89.1	77	85	90	1578.9	4637.3	1262.7	
	Scrub/Shrub	36.6	37.7	57.1	131.3	30	48	65	1096.7	1809.9	3709.0	
M3 Total		85.0	148.1	91.6	324.7	M3 Composite CN						66
M4	Evergreen Forest	2.1	0.3	1.7	4.1	30	55	70	63.1	15.2	117.1	
	Grassland	2.9	0.1	3.5	6.6	39	61	74	113.1	8.9	261.2	
	High Intensity Developed	14.7	0.1	7.0	21.7	89	92	94	1306.2	6.4	656.1	
	Low Intensity Developed	20.1	0.2	11.3	31.6	77	85	90	1547.9	16.3	1015.4	
	Scrub/Shrub	25.1	0.0	25.5	50.6	30	48	65	752.3	0.6	1655.4	
M4 Total		64.9	0.7	48.9	114.5	M4 Composite CN						66
M5	Bare Land		0.2	0.0	0.2	72	82	87	0.0	18.2	0.0	
	Evergreen Forest		56.1	27.3	83.3	30	55	70	0.0	3083.8	1909.6	
	Grassland		4.2	0.0	4.2	39	61	74	0.0	254.6	0.0	
	High Intensity Developed		1.2	0.0	1.2	89	92	94	0.0	113.0	0.0	
	Low Intensity Developed		27.5	0.0	27.5	77	85	90	0.0	2336.1	0.0	
	Scrub/Shrub		40.4	163.1	203.6	30	48	65	0.0	1940.8	10603.9	
M5 Total			129.6	190.4	320.0	M5 Composite CN						63
M6	Bare Land		0.2	0.0	0.2	72	82	87	0.0	18.2	0.0	
	Evergreen Forest		18.2	7.8	25.9	30	55	70	0.0	999.4	543.3	
	Grassland		9.1	0.6	9.7	39	61	74	0.0	554.6	42.6	
	High Intensity Developed		2.9	1.5	4.5	89	92	94	0.0	269.8	144.3	
	Low Intensity Developed		67.7	5.2	72.9	77	85	90	0.0	5754.8	471.5	
	Scrub/Shrub		40.0	72.8	112.8	30	48	65	0.0	1919.1	4730.1	
M6 Total			138.1	87.9	226.0	M6 Composite CN						68
M7	Bare Land	0.4		1.1	1.5	72	82	87	32.0	0.0	95.3	
	Evergreen Forest	13.7		1.8	15.5	30	55	70	411.9	0.0	123.5	
	Grassland	2.8		24.1	26.9	39	61	74	110.2	0.0	1780.6	
	High Intensity Developed	9.5		5.2	14.7	89	92	94	843.2	0.0	489.6	
	Low Intensity Developed	13.9		22.8	36.7	77	85	90	1072.0	0.0	2049.7	
	Scrub/Shrub	25.7		36.4	62.1	30	48	65	771.4	0.0	2366.4	
M7 Total		66.1		91.3	157.4	M7 Composite CN						64
M8	Bare Land		0.4	0.0	0.4	72	82	87	0.0	29.9	1.9	
	Evergreen Forest			1.7	1.7	30	55	70	0.0	0.0	117.6	
	Grassland		0.7	1.3	2.1	39	61	74	0.0	44.1	98.4	
	High Intensity Developed		0.5	3.2	3.7	89	92	94	0.0	44.3	300.0	
	Low Intensity Developed		10.4	7.8	18.2	77	85	90	0.0	881.5	703.6	
	Scrub/Shrub		1.0	8.0	9.0	30	48	65	0.0	47.0	523.2	
M8 Total			12.9	22.1	35.0	M8 Composite CN						80
M9	Bare Land	0.4		0.5	0.8	72	82	87	25.7	0.0	39.4	
	Evergreen Forest	2.3		1.2	3.5	30	55	70	68.7	0.0	87.4	
	Grassland	2.7		2.4	5.1	39	61	74	106.2	0.0	176.6	
	High Intensity Developed	0.5		5.3	5.8	89	92	94	40.7	0.0	497.7	
	Low Intensity Developed	5.0		21.2	26.1	77	85	90	382.1	0.0	1903.9	
	Scrub/Shrub	6.2		23.9	30.0	30	48	65	184.9	0.0	1551.0	
M9 Total		17.0		54.4	71.4	M9 Composite CN						71
P1	Evergreen Forest		27.6	8.6	36.1	30	55	70	0.0	1517.2	598.6	
	Grassland		0.2	12.0	12.2	39	61	74	0.0	12.0	891.1	
	Scrub/Shrub		11.9	365.5	377.5	30	48	65	0.0	573.4	23759.2	
P1 Total			39.7	386.1	425.8	P1 Composite CN						64

Table 4-3 (Continued). Calculation of Composite Curve Numbers for Ala Wai Watershed

Calculation of Composite Curve Numbers for Ala Wai Watershed (Continued)

Sum of WS_Acre		New HydroGrp HSG (All D and blank to C)				Curve Number			Area x CN			Composite
SUB-BASIN	LAND USE	A	B	C	Total	A	B	C	A	B	C	CN
P2	Bare Land		0.4	0.0	0.4	72	82	87	0.0	29.9	0.0	
	Evergreen Forest		37.9	51.3	89.2	30	55	70	0.0	2084.9	3590.3	
	Grassland		0.9	19.3	20.1	39	61	74	0.0	52.6	1425.9	
	High Intensity Developed		1.5	0.0	1.5	89	92	94	0.0	133.5	0.0	
	Low Intensity Developed		9.1	0.0	9.1	77	85	90	0.0	769.4	1.4	
	Scrub/Shrub		29.8	513.0	542.8	30	48	65	0.0	1432.6	33343.	
P2 Total			79.5	583.6	663.0	P2 Composite CN						65
P3	Bare Land		0.1	0.0	0.1	72	82	87	0.0	11.7	0.0	
	Evergreen Forest		65.4	36.3	101.7	30	55	70	0.0	3599.5	2538.8	
	Grassland		6.7	3.5	10.2	39	61	74	0.0	407.5	258.1	
	High Intensity Developed		3.8	0.0	3.8	89	92	94	0.0	351.0	0.0	
	Low Intensity Developed		9.9	0.6	10.6	77	85	90	0.0	841.7	58.5	
	Scrub/Shrub		43.1	138.1	181.2	30	48	65	0.0	2070.0	8973.6	
P3 Total			129.1	178.5	307.6	P3 Composite CN						62
P4	Bare Land		1.2	1.7	2.9	72	82	87	0.0	95.5	150.2	
	Evergreen Forest		5.2	31.6	36.8	30	55	70	0.0	284.5	2215.1	
	Grassland		6.4	9.7	16.1	39	61	74	0.0	390.0	714.8	
	High Intensity Developed		17.8	12.2	30.1	89	92	94	0.0	1642.0	1147.2	
	Low Intensity Developed		26.6	25.1	51.8	77	85	90	0.0	2262.4	2263.2	
	Scrub/Shrub		12.4	138.0	150.3	30	48	65	0.0	593.0	8969.7	
P4 Total			69.5	218.4	287.9	P4 Composite CN						72
P5	Bare Land		0.2	0.1	0.4	72	82	87	0.0	18.2	12.4	
	Evergreen Forest		6.9	6.9	13.9	30	55	70	0.0	381.6	486.4	
	Grassland		1.8	4.2	6.0	39	61	74	0.0	107.7	310.4	
	High Intensity Developed		3.9	5.7	9.6	89	92	94	0.0	362.7	534.8	
	Low Intensity Developed		19.6	41.4	61.0	77	85	90	0.0	1667.5	3721.7	
	Scrub/Shrub		10.7	94.2	104.9	30	48	65	0.0	514.0	6121.2	
P5 Total			43.2	152.5	195.7	P5 Composite CN						73
P6	Bare Land		1.3	5.3	6.6	72	82	87	0.0	106.2	462.5	
	Evergreen Forest			1.0	1.0	30	55	70	0.0	0.0	73.1	
	Grassland		3.4	28.5	31.9	39	61	74	0.0	206.4	2109.5	
	High Intensity Developed		35.3	172.3	207.6	89	92	94	0.0	3248.2	16199.	
	Low Intensity Developed		19.2	79.2	98.4	77	85	90	0.0	1631.9	7131.8	
	Scrub/Shrub		1.3	89.4	90.7	30	48	65	0.0	60.4	5813.3	
P6 Total			60.4	375.9	436.3	P6 Composite CN						85
P7	Bare Land		1.0	0.0	1.0	72	82	87	0.0	84.6	0.0	
	Cultivated Land			0.2	0.2	77	86	91	0.0	0.0	20.2	
	Evergreen Forest		0.2	0.2	0.4	30	55	70	0.0	12.2	15.6	
	Grassland		12.1	1.3	13.4	39	61	74	0.0	738.4	93.7	
	High Intensity Developed		145.3	50.3	195.6	89	92	94	0.0	13369.	4724.6	
	Low Intensity Developed		47.7	13.6	61.2	77	85	90	0.0	4052.5	1221.0	
	Scrub/Shrub		12.2	0.6	12.8	30	48	65	0.0	585.6	41.8	
P7 Total			218.6	66.2	284.7	P7 Composite CN						88
W1	Evergreen Forest	0.6		5.4	6.0	30	55	70	18.7	0.0	377.5	
	Grassland	0.2		8.8	9.0	39	61	74	8.7	0.0	652.2	
	High Intensity Developed	9.4		60.2	69.6	89	92	94	833.4	0.0	5661.4	
	Low Intensity Developed	3.5		15.2	18.8	77	85	90	271.3	0.0	1371.0	
	Water			0.0	0.0	98	98	98		0.0	2.9	
W1 Total		13.7		89.7	103.4	W1 Composite CN						89
W2	Evergreen Forest	0.1		1.0	1.1	30	55	70	4.3	0.0	69.0	
	Grassland			0.8	0.8	39	61	74	0.0	0.0	56.5	
	High Intensity Developed	8.4		63.0	71.4	89	92	94	749.3	0.0	5918.8	
	Low Intensity Developed	1.0		8.5	9.4	77	85	90	74.7	0.0	762.1	
	Scrub/Shrub			0.0	0.0	30	48	65	0.0	0.0	0.1	
	Water			0.0	0.0	98	98	98		0.0	2.2	
W2 Total		9.5		73.2	82.7	W2 Composite CN						92
W3	Bare Land	0.3		0.0	0.3	72	82	87	20.7	0.0	0.0	
	Evergreen Forest	0.9		0.9	1.7	30	55	70	26.4	0.0	60.3	
	Grassland	0.1	0.0	0.0	0.1	39	61	74	3.8	0.7	0.0	
	High Intensity Developed	57.7	0.1	41.6	99.5	89	92	94	5137.	12.1	3910.8	
	Low Intensity Developed	4.3		5.5	9.9	77	85	90	334.2	0.0	497.4	
	Water			0.6	0.6	98	98	98		0.0	54.4	
W3 Total		63.3	0.1	48.5	112.0	W3 Composite CN						90
Grand Total		2053.8	3344.	4978.	10377.3	Ala Wai Watershed Composite CN						70

Table 4-3 (Continued). Calculation of Composite Curve Numbers for Ala Wai Watershed

4.4 Mānoa-Pālolo Model Calibration

The final HEC-HMS model for the Ala Wai Watershed consisted of 38 sub-basins. The model used the SCS runoff curve number method as the loss method to be consistent with the previous Mānoa Watershed Project hydrologic study. The model for the Ala Wai Watershed used the Clark Unit Hydrograph as the transform method for the sub-basins that are not fully urbanized. The Clark Unit Hydrograph was used as the transform method for the sub-basins in Makiki Valley (K1-K4), Mānoa Valley (M1 to M14), and Pālolo Valley (P1 to P7). The urbanized sub-basins of lower Makiki, Ala Wai Canal, and Waikīkī applied the Kinematic Wave Transform Method. Because there are insufficient rainfall and stream flow data in the low-lying areas of the Ala Wai Watershed, it was difficult to calibrate the sub-basin parameters within in the Ala Wai Canal and Waikīkī sub-watersheds. Most of the parameters of the Kinematic Wave Transform Method were based on physical measurements; it is assumed that the peak discharges of the urbanized sub-basins are correct. The actual calibration models are those of Mānoa and Pālolo valleys, a pilot calibration model for Makiki valley, and a reservoir calibration model for Ala Wai Canal. This last model represents the calibration for the entire watershed. Figure 4-4 shows the calibration model layout for the Ala Wai Watershed.

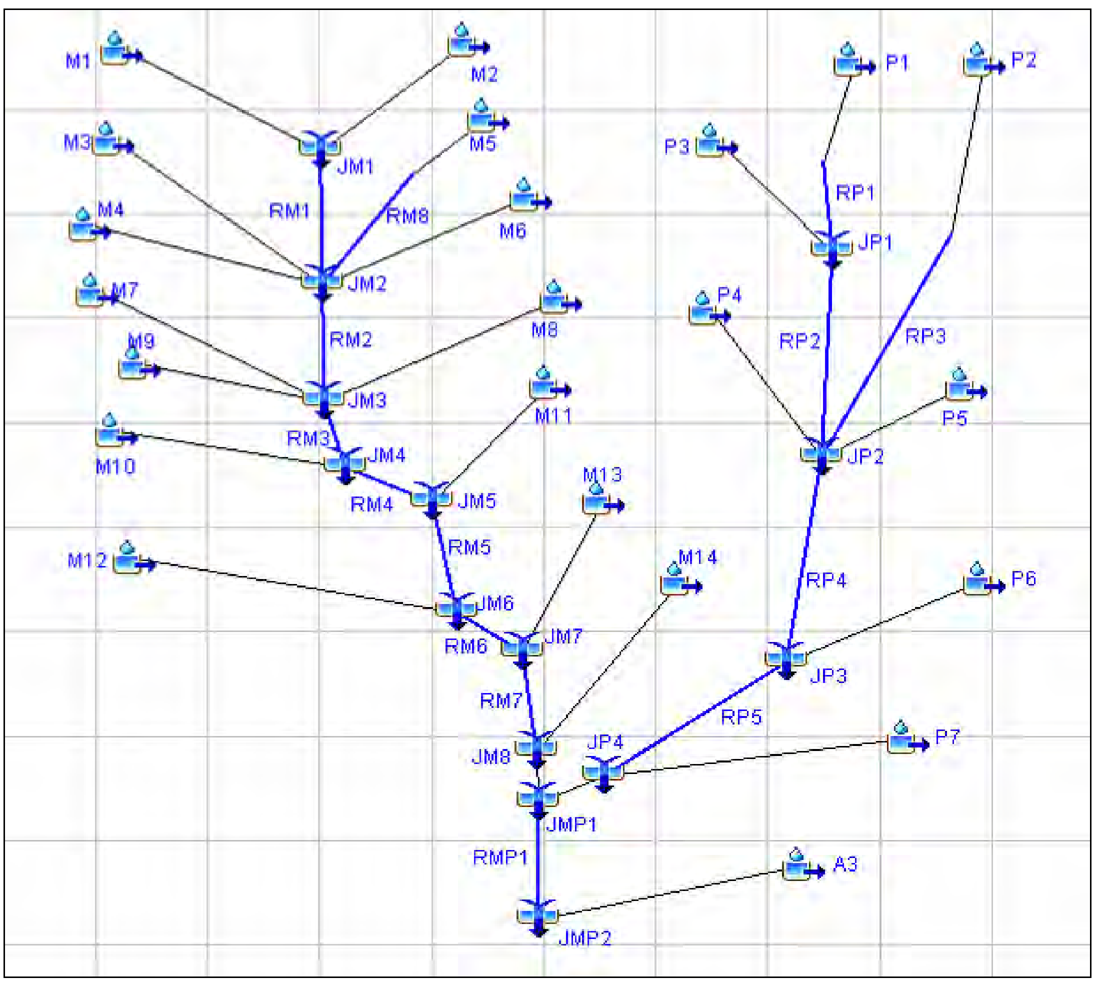

Figure 4-4. HEC-HMS Mānoa-Pālolo Calibration Model Layout

4.4.1 October 2004 Storm Calibration for Mānoa-Pālolo Area

The calibration for the storm of October 30, 2004, was based on the method used in the previous Mānoa Watershed Project hydrologic study. The calibration parameters used for the Mānoa sub-watershed in the Mānoa Watershed Project hydrologic study were used for the HEC-HMS model in the Ala Wai Watershed hydrologic study. The gage weights for sub-basins in Mānoa valley were the same as those used in the Mānoa Watershed Project described earlier. The main task of the calibration for the Ala Wai Watershed hydrologic study focused on the Pālolo sub-watershed and the area downstream of Kānewai Field gage, to the USGS stream gage 16247100. This stream gage is located on Kaimukī High School and had full stream flow records for the event. Gage weights were used for calibration purposes. The Thiessen polygon method was initially applied to determine the gage weight for each sub-basin. Figure 4-5 shows the Thiessen polygons for the October 2004 storm for the Ala Wai Watershed. The Thiessen polygon method does not account for orthographic rainfall effect in mountain areas. After taking into consideration the rainfall pattern, data quality, and storm movement and distribution, the final gage weights and relevant 24-hour rainfall of the October 2004 storm for each sub-basin were determined as shown in Table 4-4. (Note: 'MP' is used to abbreviate the Mānoa-Pālolo area.)

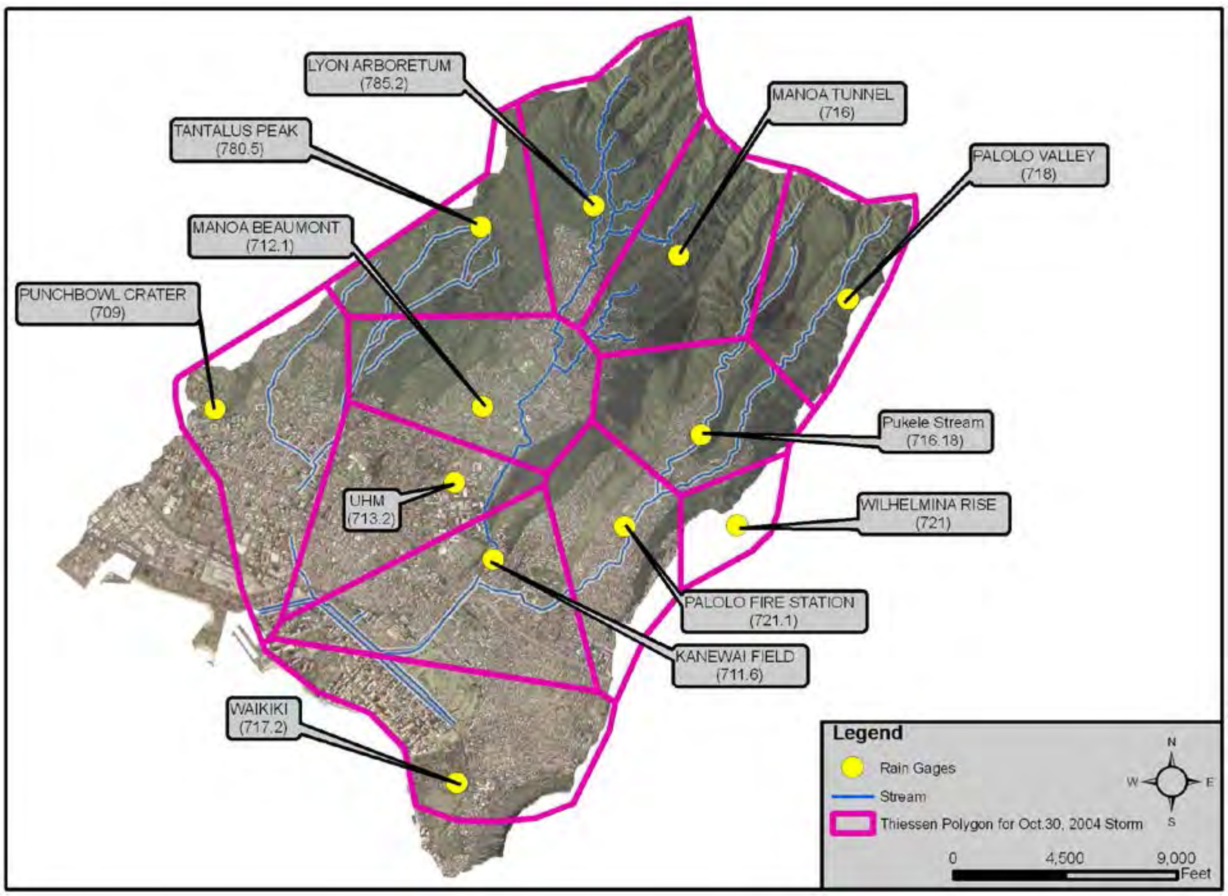

Figure 4-5. Rain Gages and Thiessen Polygons for the October 30, 2004

Meteorological Model: Gage Weights for October 30, 2004, Storm for MP

Gage weights	Thiessen Polygon (Gages in red are real time recording)											
Sub-basin	**Lyon Arboretum**	**Manoa Tunnel**	**Kanewai**	**Manoa Beaumont**	**UHM**	**Palolo Fire Sta**	**Palolo Valley**	**Pūkele**	**Tantalus Peak**	**Waikiki**	**Wilhemina Rise**	**24hr Rain (in)**
ID	785.2	716	711.6	712.1	713.2	721.1	718	Pukele	780.5	717.2	721	
Total Rainfall (in)	10.08	11.14	1.67	4.62	2.4	2.13	6.21	4.07	7.8	0.05	1.64	(in)
A3			0.7			0.2				0.1		1.60
M1	0.8	0.2										10.29
M2	0.3	0.5					0.2					9.84
M3	0.6	0.2							0.2			9.84
M4	0.5	0.2							0.3			9.61
M5	0.3	0.4					0.3					9.34
M6	0.4	0.3		0.1				0.2				8.65
M7	0.4			0.3					0.3			7.76
M8	0.3			0.4				0.3				6.09
M9	0.3			0.4					0.3			7.21
M10	0.3			0.4					0.3			7.21
M11	0.3			0.4				0.3				6.09
M12			0.1	0.5	0.4							3.44
M13			0.3	0.2	0.2	0.3						2.57
M14			0.5			0.5						1.9
P1		0.3					0.5	0.2				7.26
P2		0.1					0.6	0.3				6.06
P3		0.1					0.3	0.6				5.42
P4		0.2				0.1		0.7				5.29
P5						0.1		0.5			0.4	2.9
P6						0.9					0.1	2.08
P7			0.5			0.5						1.9

Table 4-4. Meteorological Model: Gage Weights for October 30, 2004, Storm for MP

The HEC-HMS meteorological model's parameters were calibrated using the October 30, 2004, storm data. Table 4-5 lists the final parameters for the HEC-HMS model in the Mānoa and Pālolo sub-watersheds. The parameters of the calibrated times of concentration are close to those calculated using the TR-55 method. The meteorological model used storm hydrographs for calibration and frequency based rainfall to compute the synthetic flood events.

	Loss Method --- SCS Curve Number		Transform--Clark Unit Hydrograph	
Sub-basin	Initial Abstraction (inch)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
A3 (Plane 1)	0.75	83	Kinematic Wave Transform	
A3 (Plane 2)	0.10	98		
M1	0.60	62	0.24	0.42
M10	0.60	76	0.26	0.60
M11	0.60	75	0.50	0.30
M12	0.30	73	0.25	0.65
M13	0.60	68	0.27	0.40
M14	1.00	84	0.15	0.30
M2	0.60	64	0.23	1.10
M3	0.60	69	0.25	0.70
M4	0.60	73	0.23	0.80
M5	0.60	63	0.31	0.90
M6	0.60	68	0.25	0.85
M7	0.60	71	0.19	1.50
M8	0.60	80	0.16	1.80
M9	0.60	75	0.17	1.50
P1	2.20	64	0.10	0.40
P2	1.20	65	0.30	0.55
P3	3.20	62	0.10	0.68
P4	1.20	72	0.10	0.30
P5	1.20	73	0.16	0.30
P6	1.20	85	0.10	0.25
P7	1.20	88	0.18	0.30

Table 4-5. Calibrated Model Parameters for October 2004 Storm for MP

At junctions JP1 and JMP2, the observed rainfall from the October 2004 storm and the modeled stream flows are shown in Figures 4-6 and 4-7. The modeled peak flows occur slightly after the observed peak flows; and the peak flow for junction JP1, Pūkele Stream gage, was modeled at a higher amount than the observed peak flow in 2004. The time of concentration values may be too high in this case. For the October 2004 storm, real-time data from M2, partial data from JM3, partial real-time data from JM7, real-time data from JP1, peak flow data from JP3, and continuous data from JMP2 were used. Because the HEC-HMS model was calibrated using the October 2004 storm data in the Mānoa Watershed Project hydrologic study (Oceanit 2008), the parameters for all Mānoa sub-basins except M14 in the Ala Wai Watershed hydrologic study were kept the same as they were in the Mānoa Watershed Project study.

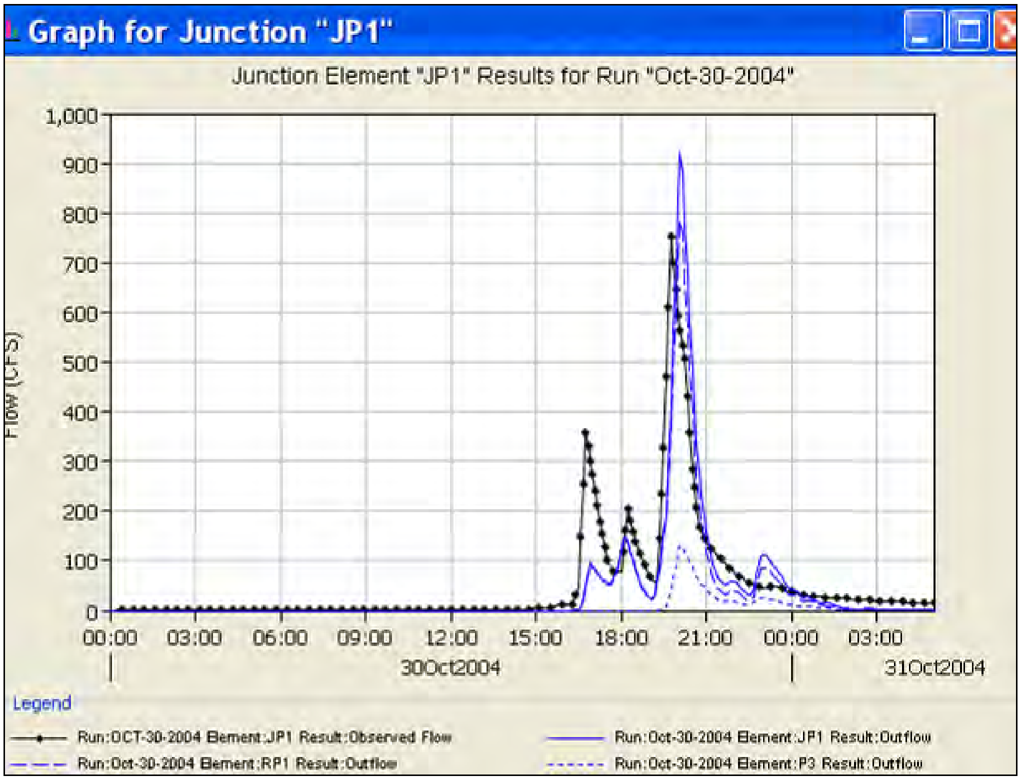

Figure 4-6. Observed and Modeled Stream Flows at Junction JP1 (Pūkele Gage [2440]) October 2004 Storm

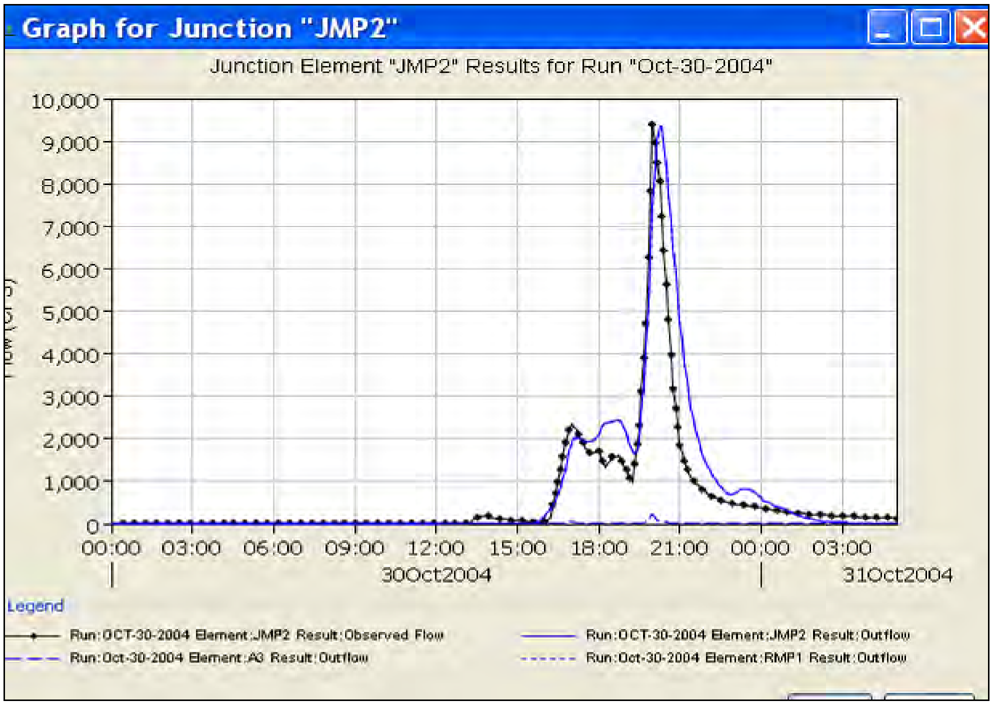

Figure 4-7. Observed and Modeled Flows at Junction JMP2 (Mānoa-Pālolo Gage [2471]) October 2004 Storm

4.4.2 December 1967 Storm Calibration for Mānoa-Pālolo Area

The calibration for the storm of December 17–18, 1967, was based on the method used in the previous Mānoa Watershed Project hydrologic study. The calibration parameters in the Mānoa Watershed Project study for the Mānoa sub-watershed were not changed. The gage weights for sub-basins in the Mānoa sub-watershed were the same as that in the Mānoa Watershed Project study. The Thiessen polygon method was initially applied to determine the gage weight for each sub-basin. Figure 4-8 shows the Thiessen polygons for the December 1967 storm for the Ala Wai Watershed. After taking into consideration the rainfall pattern, data quality, and storm movement and distribution, the final gage weights and relevant 24-hour rainfall of the December 1967 storm for each sub-basin were determined as shown in Table 4-6. (Note: 'MP' is used to abbreviate the Mānoa-Pālolo area.)

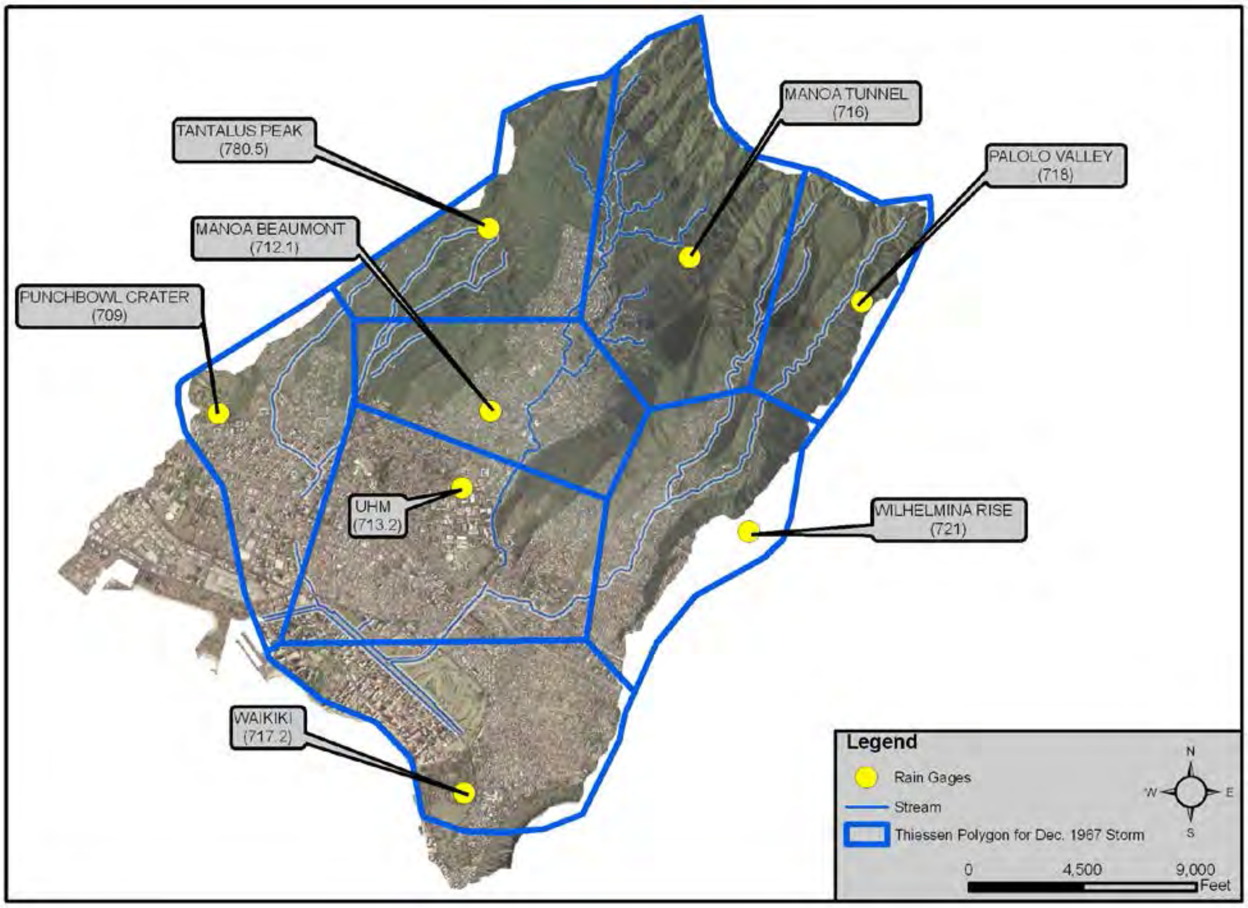

Figure 4-8. Rain Gages and Thiessen Polygons for December 1967 Storm for MP

The gage weights for the December 1967 storm, shown in Table 4-6, were calculated by considering the Thiessen polygons shown in Figure 4-8, the rainfall pattern, and the storm movement and distribution.

Meteorological Model: Gage Weights for December 17–18, 1967, Storm for MP								
Gage weights	**Thiessen Polygon**							
Sub-Basin	**Mānoa Tunnel**	**Mānoa Beaumont**	**UHM**	**Pālolo Valley**	**Tantalus Peak**	**Waikīkī**	**Wilhemina Rise**	**24-hr Rain (in)**
ID	716	712.1	713.2	718	780.5	717.2	721	
Total Rainfall (in)	10.42	9.43	9.5	10.88	8.1	8.21	9.56	
A3			0.6		0.1	0.3		8.97
M1	0.4				0.6			9.03
M2	0.6			0.2	0.2			10.05
M3	0.2	0.2			0.6			8.83
M4	0.2	0.2			0.6			8.83
M5	0.5			0.3	0.2			10.09
M6	0.3	0.5			0.2			9.46
M7		0.6			0.4			8.90
M8		0.8			0.2			9.16
M9		0.7			0.3			9.03
M10		0.8			0.2			9.16
M11		0.6	0.2		0.2			9.18
M12		0.4	0.5		0.1			9.33
M13		0.2	0.7		0.1			9.35
M14		0.2	0.6		0.1		0.1	9.35
P1	0.5			0.4	0.1			10.37
P2				0.7	0.1		0.2	10.34
P3	0.3			0.4	0.1		0.2	10.2
P4	0.3				0.1		0.6	9.68
P5					0.1		0.9	9.41
P6			0.2		0.1		0.7	9.4
P7			0.45		0.1	0.15	0.3	9.18

Table 4-6. Meteorological Model: Gage Weights for December 17–18, 1967, Storm for MP

The meteorological model used storm hydrographs for calibration and frequency based rainfall to compute the synthetic flood events. For creating the peak discharges for various return periods, the frequency storm with an intensity position at 50% was used in computing the peaks and hydrographs. Table 4-7 lists the final parameters for the HEC-HMS model in the Mānoa and Pālolo sub-watersheds. The parameters of the calibrated time of concentrations are close to those calculated using the TR-55 method. At junctions JP1, JP3, and JMP2, the modeled stream flows for the December 1967 storm show a series of stream flow peaks as shown in Figures 4-9 to 4-11. For the December 1967 storm, peak flow data from M2, data from JP1, peak flow data from JP3, and continuous data from JMP2 were used.

	Loss Method --- SCS Curve Number		Transform--Clark Unit Hydrograph	
Sub-basin	Initial Abstraction (inches)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
A3 (Plane 1)	1.50	83	Kinematic Wave Transform	
A3 (Plane 2)	0.15	98		
M1	0.70	62	0.22	0.30
M10	0.70	76	0.26	0.25
M11	0.70	75	0.19	0.25
M12	0.70	73	0.26	0.22
M13	0.70	68	0.26	0.30
M14	1.80	84	0.10	0.68
M2	0.50	64	0.22	0.22
M3	0.70	69	0.22	0.30
M4	0.70	73	0.22	0.30
M5	0.70	63	0.23	0.30
M6	0.70	68	0.22	0.30
M7	0.70	71	0.18	0.30
M8	0.70	80	0.15	0.30
M9	0.70	75	0.17	0.30
P1	1.20	64	0.21	0.30
P2	1.80	65	0.30	0.20
P3	1.20	62	0.16	0.25
P4	0.72	72	0.25	0.23
P5	0.65	73	0.30	0.34
P6	0.73	85	0.24	0.31
P7	1.80	88	0.10	0.80

Table 4-7. Calibrated Model Parameters for December 1967 Storm for MP

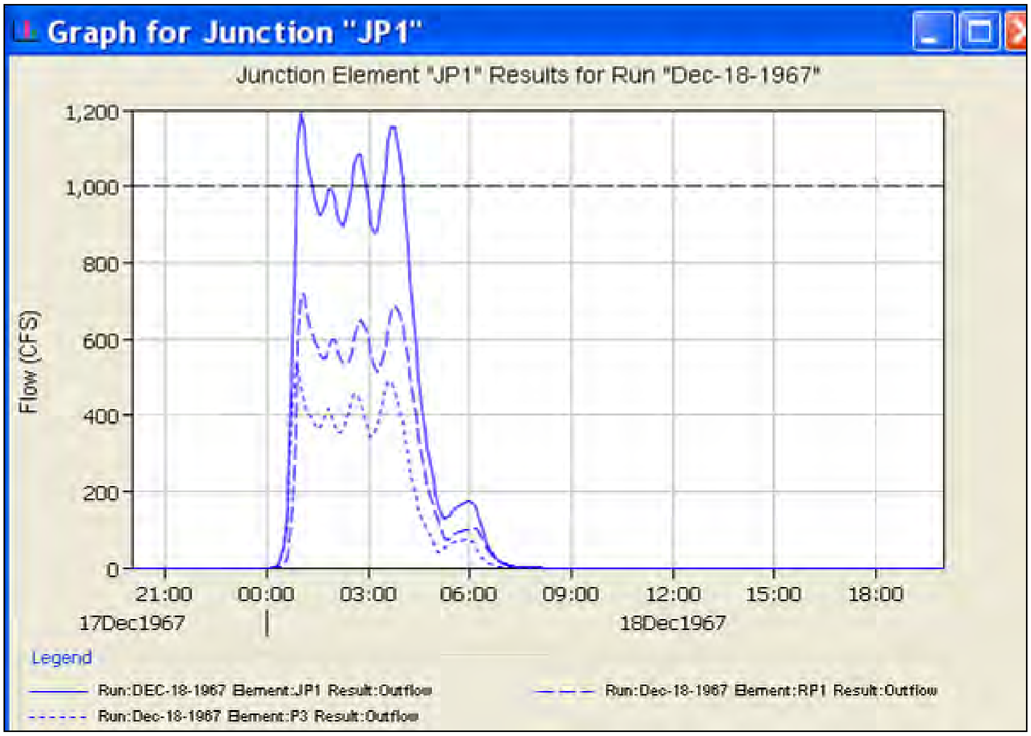

Figure 4-9. Modeled Stream Flows at Junction JP1 (Pūkele Gage [2440]) December 1967 Storm

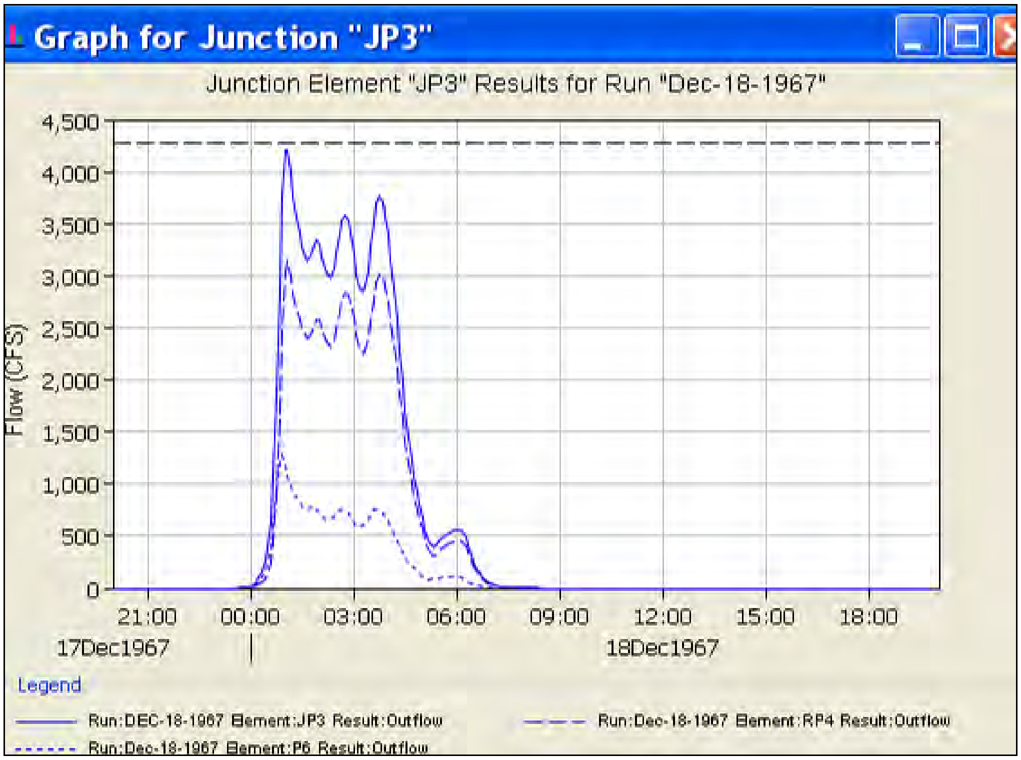

Figure 4-10. Modeled Stream Flows at JP3 (USGS Pālolo Gage [16247000]) December 1967 Storm

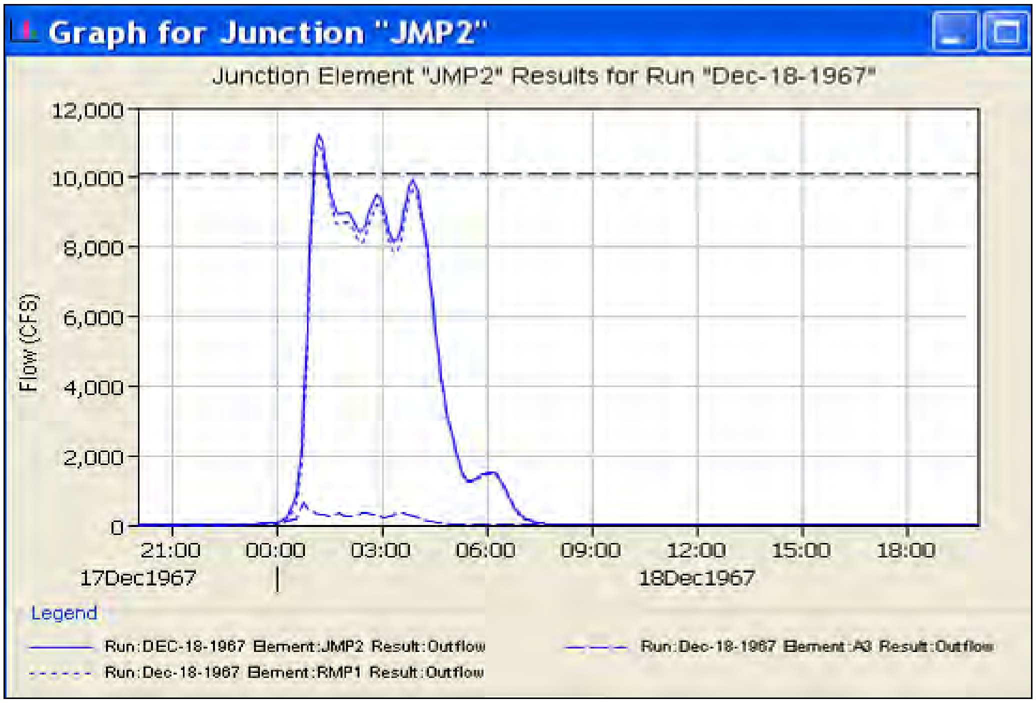

Figure 4-11. Modeled Stream Flows at JMP2 (USGS Stream Gage [16247100]) December 1967 Storm

4.4.3 March 2006 Storm Calibration for Mānoa-Pālolo Area

The Thiessen polygon method was initially applied to determine the gage weight. Figure 4-12 shows the Thiessen polygons for the March 31, 2006, storm for the Ala Wai Watershed. After taking into consideration the rainfall pattern, data quality, and storm movement and distribution, the final gage weights and relevant 24-hour rainfall of the March 2006 storm for each sub-basin were determined as shown in Table 4-8. The March 31, 2006, storm is a significant example because the storm produced a small amount of rain that generated a large amount of runoff because the soils in the study area were already saturated from six weeks of heavy rains. (Note: 'MP' is used to abbreviate the Mānoa-Pālolo area.)

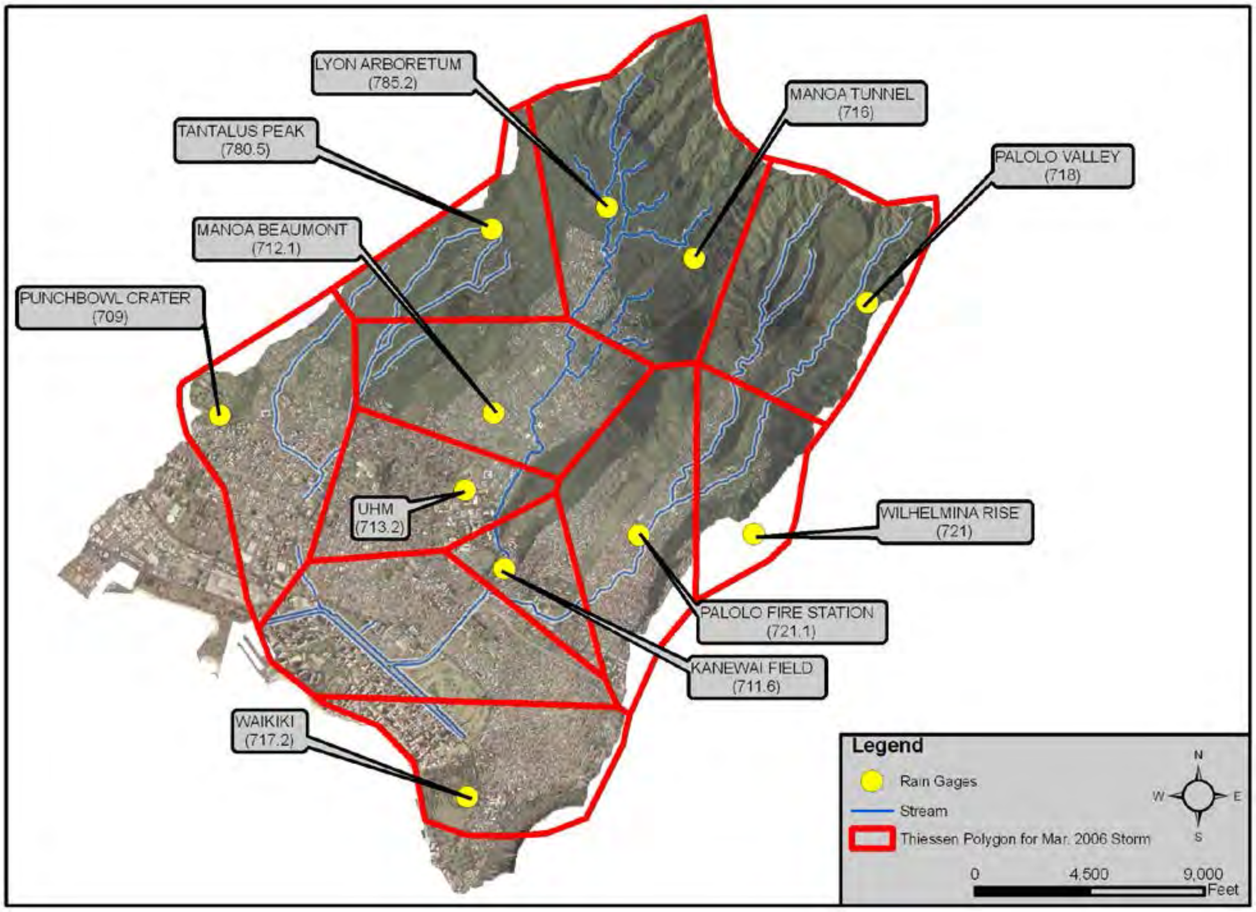

Figure 4-12. Rain Gages and Thiessen Polygons for March 2006 Storm for MP

Meteorological Model: Gage Weights for March 31, 2006 Storm for MP

Gage weights	Thiessen Polygon									
Sub-Basin	**Lyon Arboretum**	**Kānewai**	**Mānoa Beaumont**	**UHM**	**Pālolo Fire Stn.**	**Pālolo Valley**	**Tantalus Peak**	**Wilhemina Rise**	**H-1 at Kapiolani**	**24hr Rain (in)**
ID	785.2	711.6	712.1	713.2	721.1	718	780.5	721	711.7	
Total Rainfall (in)	3.35	3.49	3.25	4.75	3.00	2.84	2.60	3.49	3.53	
A3		0.1			0.1				0.8	3.47
M1	0.9						0.1			3.27
M2	0.9					0.1				3.30
M3	0.6		0.3				0.1			3.25
M4	0.1		0.6				0.3			3.07
M5	0.9				0.1					3.31
M6	0.1		0.7		0.2					3.21
M7	0.1		0.6				0.3			3.07
M8			0.8		0.2					3.20
M9	0.1		0.8	0.1						3.41
M10		0.1	0.8				0.1			3.21
M11			0.7	0.1	0.2					3.35
M12		0.2	0.3	0.5						4.05
M13		0.5		0.2	0.3					3.60
M14		0.5			0.4				0.1	3.30
P1	0.2					0.8				2.94
P2	0.1					0.7		0.2		3.02
P3	0.1				0.1	0.5		0.3		3.10
P4					0.6	0.1		0.3		3.13
P5					0.2			0.8		3.39
P6		0.1			0.8			0.1		3.10
P7		0.6			0.3				0.1	3.35

Table 4-8. Meteorological Model: Gage Weights for March 2006 Storm for MP

The HEC-HMS meteorological model's parameters were calibrated using the March 31, 2006, storm data. Table 4-9 lists the final parameters for the HEC-HMS model in the Mānoa and Pālolo sub-watersheds. The parameters of the calibrated time of concentrations are close to those calculated using the TR-55 method. The meteorological model used storm hydrographs for calibration and frequency based rainfall to compute the synthetic flood events.

	Loss Method --- SCS Curve Number		Transform Method --- Clark Unit Hydrograph	
Sub-basin	Initial Abstraction (inches)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
A3 (Plane 1)	0	92	Kinematic Wave Transform	
A3 (Plane 2)	0	98		
M1	0	88	0.20	0.10
M10	0	92	0.18	0.10
M11	0	92	0.10	0.10
M12	0	92	0.20	0.10
M13	0	90	0.10	0.10
M14	0	90	0.12	0.10
M2	0	70	0.32	0.12
M3	0	92	0.20	0.10
M4	0	92	0.10	0.10
M5	0	72	0.20	0.10
M6	0	75	0.15	0.10
M7	0	80	0.15	0.10
M8	0	92	0.10	0.10
M9	0	92	0.10	0.10
P1	0	64	0.10	0.10
P2	0	65	0.10	0.10
P3	0	62	0.10	0.10
P4	0	72	0.10	0.10
P5	0	73	0.10	0.10
P6	0	85	0.10	0.11
P7	0	90	0.10	0.10

Table 4-9. Calibrated Model Parameters for March 2006 Storm

The modeled stream flow for the March 2006 storm in M2 and at JMP2 show a small flow peak flow followed by a higher peak flow, as shown in Figures 4-13 and 4-14. The modeled peak flows are higher and earlier than the observed flows; however, the highest peaks match well. Due to the extremely saturated soil within the study area during this storm, the sub-basins' curve numbers were allowed to change to match the peak at JMP2 for calibration.

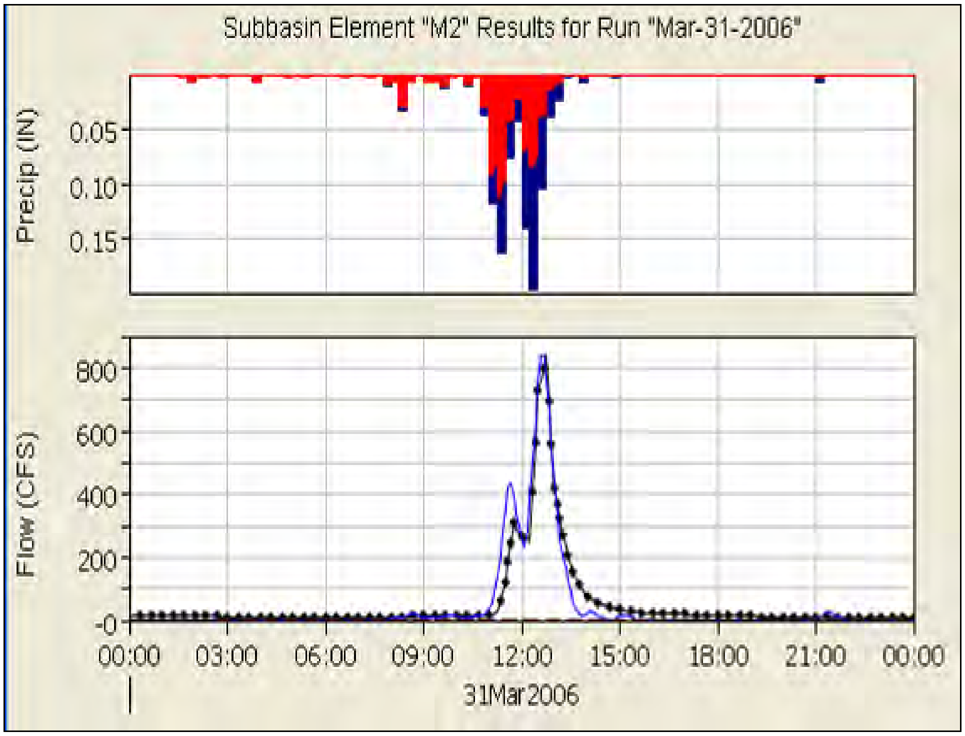

Figure 4-13. Observed and Modeled Stream Flows at Waiakeakua Stream (Sub-basin M2), March 2006 Storm

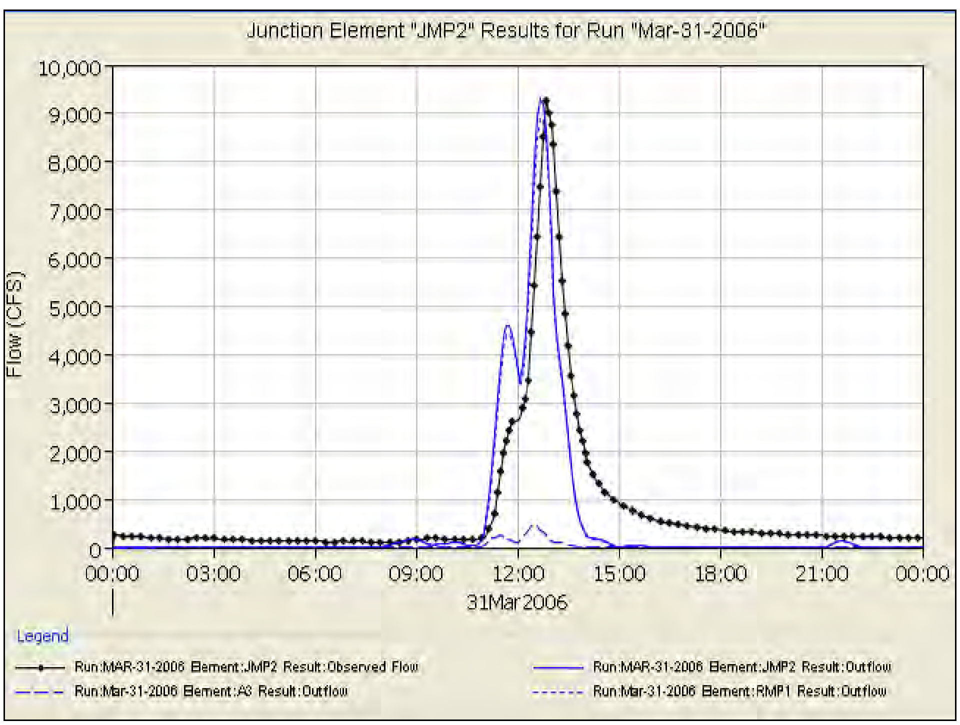

Figure 4-14. Observed and Modeled Stream Flows at JMP2 (USGS Stream Gage [16247100]), March 2006 Storm

4.4.4 Final Loss and Transform Parameters for Mānoa-Pālolo Area

The loss method was determined by using the NRCS runoff CN method to take advantage of the results from the Mānoa Watershed Project hydrologic study. The parameters of initial abstraction were optimized for the Waiakeakua sub-basin and then were assigned to all the other sub-basins. Impervious parameters were set to zero because the percentage of the sub-basin that is impervious is specified in the CN. The optimization was used in each individual calibration (see Section 4.2). The final model parameters were the weighted average ones.

There is more confidence with the storms of October 30, 2004, and December 17–18, 1967, and less confidence with the storm of March 31, 2006. More weighting values were given to the calibrated parameters of the storm events of October 2004 and December 1967. The calibrated parameters of the October 2004 and December 1967 storm events were assigned twice the weight of the calibrated parameters for the March 31, 2006, storm. The finalized calibrated parameters of the HEC-HMS model were weighted as (2*2004 + 2*1967 + 1* 2006)/5. The weighted averaged loss method and transform method parameters for the Mānoa-Pālolo area are listed in Tables 4-10 and 4-11.

Curve Number Loss Method Calibration: Manoa-Palolo basin model								
	October-30-2004		December-18-1967		March-31-2006		Weighted Average	
Sub-basin	Initial Abstraction (inch)	Curve Number	Initial Abstraction (inch)	Curve Number	Initial Abstraction (inch)	Curve Number	Initial Abstraction (inch)	Curve Number
A3 (Plane 1)	0.75	83	1.50	83	0	92	**0.90**	**85**
A3 (Plane 2)	0.10	98	0.15	98	0	98	**0.10**	**98**
M1	0.60	62	0.70	62	0	88	**0.52**	**67**
M10	0.60	76	0.70	76	0	92	**0.52**	**79**
M11	0.60	75	0.70	75	0	92	**0.52**	**78**
M12	0.30	73	0.70	73	0	92	**0.40**	**77**
M13	0.60	68	0.70	68	0	90	**0.52**	**72**
M14	1.00	84	1.80	84	0	90	**1.12**	**85**
M2	0.60	64	0.50	64	0	70	**0.44**	**65**
M3	0.60	69	0.70	69	0	92	**0.52**	**74**
M4	0.60	73	0.70	73	0	92	**0.52**	**77**
M5	0.60	63	0.70	63	0	72	**0.52**	**65**
M6	0.60	68	0.70	68	0	75	**0.52**	**69**
M7	0.60	71	0.70	71	0	80	**0.52**	**73**
M8	0.60	80	0.70	80	0	92	**0.52**	**82**
M9	0.60	75	0.70	75	0	92	**0.52**	**78**
P1	2.20	64	1.20	64	0	64	**1.36**	**64**
P2	1.20	65	1.80	65	0	65	**1.20**	**65**
P3	3.20	62	1.20	62	0	62	**1.76**	**62**
P4	1.20	72	0.72	72	0	72	**0.77**	**72**
P5	1.20	73	0.65	73	0	73	**0.74**	**73**
P6	1.20	85	0.73	85	0	85	**0.77**	**85**
P7	1.20	88	1.80	88	0	90	**1.20**	**88**

Table 4-10. Final HEC-HMS Model Loss Method Parameters

Clark Unit Hydrograph Transform Method Calibration: Manoa-Palolo basin model								
	October-30-2004		December-18-1967		March-31-2006		Average Values	
Sub-basin	T_c (Hour)	S_c (Hour)	T_c (Hour)	S_c (Hour)	T_c (Hour)	S_c (Hour)	T_c (Hour)	S_c (Hour)
M1	0.24	0.42	0.22	0.30	0.20	0.10	**0.22**	**0.31**
M2	0.23	1.10	0.22	0.22	0.32	0.12	**0.24**	**0.55**
M3	0.25	0.70	0.22	0.30	0.20	0.10	**0.23**	**0.42**
M4	0.23	0.80	0.22	0.30	0.10	0.10	**0.20**	**0.46**
M5	0.31	0.90	0.23	0.30	0.20	0.10	**0.26**	**0.50**
M6	0.25	0.85	0.22	0.30	0.15	0.10	**0.22**	**0.48**
M7	0.19	1.50	0.18	0.30	0.15	0.10	**0.18**	**0.74**
M8	0.16	1.80	0.15	0.30	0.10	0.10	**0.14**	**0.86**
M9	0.17	1.50	0.17	0.30	0.10	0.10	**0.16**	**0.74**
M10	0.26	0.60	0.26	0.25	0.18	0.10	**0.24**	**0.36**
M11	0.50	0.30	0.19	0.25	0.10	0.10	**0.30**	**0.24**
M12	0.25	0.65	0.26	0.22	0.20	0.10	**0.24**	**0.37**
M13	0.27	0.40	0.26	0.30	0.10	0.10	**0.23**	**0.30**
M14	0.15	0.30	0.10	0.68	0.12	0.10	**0.12**	**0.41**
P1	0.10	0.40	0.21	0.30	0.10	0.10	**0.14**	**0.30**
P2	0.30	0.55	0.30	0.20	0.10	0.10	**0.26**	**0.32**
P3	0.10	0.68	0.16	0.25	0.10	0.10	**0.12**	**0.39**
P4	0.10	0.30	0.25	0.23	0.10	0.10	**0.16**	**0.23**
P5	0.16	0.30	0.30	0.34	0.10	0.10	**0.20**	**0.28**
P6	0.10	0.25	0.24	0.31	0.10	0.11	**0.16**	**0.25**
P7	0.18	0.30	0.10	0.80	0.10	0.10	**0.13**	**0.46**

Table 4-11. Final HEC-HMS Transform Method Parameters
Note: T_c is the time of concentration, S_c is the storage coefficient

4.5 Makiki Model Calibration

Data from the USGS Makiki Stream Gage (16238000) at King Street Bridge was used to calibrate the Makiki HEC-HMS model. This gage measured two peaks in 2004. One peak was 487 cfs recorded on February 28, 2004, and the other peak was 1,000 cfs recorded on October 30, 2004. There were no sufficient rainfall data for February 28, 2004, so the 1,000 cfs peak on October 30, 2004, was used to calibrate the Makiki HEC-HMS model. The Thiessen polygons for October 30, 2004, in the Makiki sub-watershed can be seen in Figure 4-15, and they are the same as those for the Mānoa-Pālolo calibration of the October 30, 2004 storm. Because there was no timing rainfall gage within the Makiki sub-watershed, the Lyon Arboretum rainfall gage (785.2) was selected as the time weight gage for all sub-basins in the sub-watershed (see Table 3.1 for rainfall gage information). (Note: 'K' is used to abbreviate for the Makiki sub-watershed.)

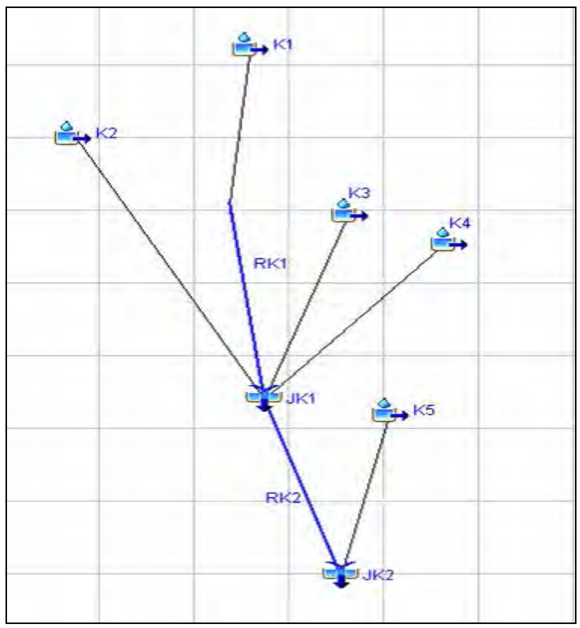

Figure 4-15. HEC-HMS Makiki Sub-Watershed Calibration Model Layout

4.5.1 October 2004 Storm Calibration for the Makiki Sub-Watershed

Due to the limited data available for the Makiki sub-watershed, the October 30, 2004, storm data were the only storm data used to calibrate the Makiki meteorological model. The calibration was based on the peak discharge of 1,000 cfs at King Street Bridge (USGS stream gage 16238000). Figure 4-5 shows the Thiessen polygons for the October 2004 storm for the Ala Wai Watershed. The Thiessen polygon method does not account for orthographic rainfall effect in mountain areas. The rainfall pattern, data quality, and storm movement and distribution were taken into consideration for the final gage weights. For the Makiki sub-watershed, the final gage weights for the 24-hour rainfall of the October 2004 storm were calculated and are given in Table 4-12.

Gage Weights for Makiki Sub-Watershed October 30, 2004, Storm						
Gage Weights	**Thiessen Polygons (Gages in red recorded)**					
Sub-Basin ID	**Lyon Arboretum**	**Mānoa Beaumont**	**UHM**	**Tantalus Peak**	**Punchbowl Crater**	**24-hr Rain (inch)**
	785.2	712.1	713.2	780.5	709	
Total Rainfall (in)	**10.08**	**4.62**	**2.4**	**7.8**	**0.05**	
K1	0.1	0.3		0.5	0.1	**6.39**
K2	0.1			0.2	0.7	**3.23**
K3	0.1	0.4	0.2		0.3	**3.62**
K4	0.1	0.4	0.4		0.1	**3.91**
K5	0.1		0.3		0.6	**2.30**

Table 4-12. Gage Weights for October 2004 Storm Makiki Sub-Watershed

The HEC-HMS meteorological model's parameters were calibrated using the October 30, 2004, storm data for the Makiki sub-watershed. Table 4-13 lists the calibrated parameters for the HEC-HMS model in the Makiki sub-watershed. The parameters of the calibrated times of concentration are close to those calculated using the TR-55 method. The meteorological model used storm hydrographs for calibration and frequency-based rainfall to compute the synthetic flood events. The final model parameters for the Makiki sub-watershed are given in Table 4-14.

Sub-basin	Initial Loss (inch)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
K1	1	42	0.18	0.45
K2	1	62	0.23	0.25
K3	1	68	0.12	0.2
K4	1	68	0.12	0.25
K5 (Plane 1)	0.7	85		
K5 (Plane 2)	0.1	98		

Table 4-13. Calibrated Parameters of Makiki Sub-Watershed

Sub-basin	Initial Loss (inch)	Curve Number	Time of Concentration (hour)	Storage Coefficient (hour)
K1	1	42	0.18	0.45
K2	1	62	0.23	0.35
K3	1	68	0.12	0.32
K4	1	68	0.12	0.35
K5 (Plane 1)	0.7	85		
K5 (Plane 2)	0.1	98		

Table 4-14. Finalized Parameters in HEC-HMS Model Makiki Sub-Watershed

The calibrated and final parameters of the HEC-HMS Model for the Makiki sub-watershed only differ by a few storage coefficients. These differences are due to the differing use and character of land in upper versus lower Makiki. The land use of the upper Makiki sub-watershed, a natural area with preservation land use, is similar to those of the upper Mānoa and Pālolo sub-watersheds. This similarity is reflected in the storage coefficient calculated. That is, the calibrated Clark Unit Hydrograph storage coefficient of the K1 sub-basin is 0.45 hour (hr), as shown in Tables 4-13 and 4-14, and the calibrated Clark Unit Hydrograph storage coefficients of sub-basins M5, P2, and M2, are 0.50, 0.32, and 0.57 hr respectively. In contrast, for lower Makiki sub-basins of K2, K3, and K4, the storage coefficients were increased slightly to match the calibrated storage coefficients in the Mānoa and Pālolo sub-watersheds. Figure 4-16 shows the modeled stream flows for JK2.

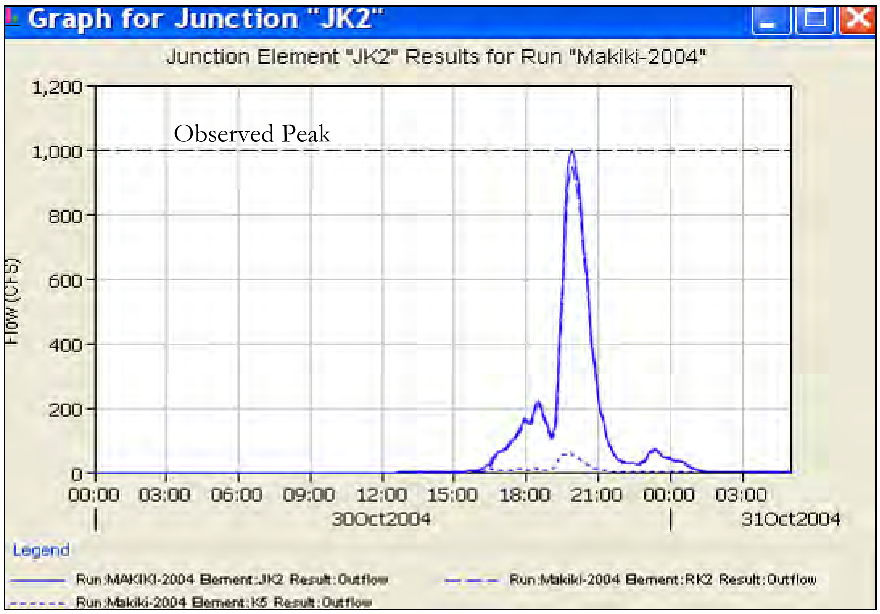

Figure 4-16. Modeled Stream Flows at JK2 (King Street Bridge, USGS stream gage 16238000)

4.6 Kinematic Wave Transform Method Parameters

The Kinematic Wave Transform Method was used for the urbanized sub-basins. The Kinematic Wave technique is widely accepted for use in urbanized runoff modeling (USACE, 2001) because the parameters for various elements constituting the model are directly related to measurable, physical basin features. Parameters such as storm drain catchment length, drainage area, roughness, slope, and channel geometry are used to define the flow of water over basin surfaces into the stream channel. For the urbanized sub-basins, two overland flow plane elements were used to represent pervious land areas such as lawns and gardens and impervious areas such as streets and roofs. In this study, a sub-basin was modeled by combining two overland planes, a collector channel, and a main channel. The lengths, slopes, and roughness coefficients of the overland flow planes were based on the average of several values within the sub-watershed. Table 4-15 lists the values of the flow planes. Urbanized watersheds typically have various storm drainage systems, man-made channels, and natural channels. To model complex urban systems in a manageable fashion, the concept of typical collector channels was employed. The collector system was formulated from average parameters, in the sub-watershed. Tables 4-16 and 4-17 summarize the values of the collector channels and main channels.

In order to use the composite runoff curve number in a kinematic wave model, the sub-watershed must be divided into its pervious and impervious components. A curve number of 98 was used for the impervious areas (USACE 1973). The following equation can be applied to calculate the adjusted pervious curve number. The adjusted pervious curve number was used as the loss rate for the pervious areas.

$$X = \frac{CNc - 98 \times f}{1 - f}$$

Where $X = Adjusted\ \ pervious\ curve\ number$

$CNc = Composite\ curve\ number$

$f = total\ \ percent\ impervious, 0 \leq f \leq 1$

Kinematic Wave Transform Flow Planes for Urbanized Sub-Basins					
Subwatershed	Intial Abstraction (inch)	CN	Area (%)	Composite CN	Adjusted Pervious CN
A1 (Plane 1)	1.00	78	70	84	78
A1 (Plane 2)	0.10	98	30		
A2 (Plane 1)	0.75	86	65	90	86
A2 (Plane 2)	0.05	98	35		
A3 (Plane 1)	0.90	83	60	89	83
A3 (Plane 2)	0.10	98	40		
A4 (Plane 1)	0.75	76	60	85	76
A4 (Plane 2)	0.10	98	40		
A5 (Plane 1)	0.75	81	60	88	81
A5 (Plane 2)	0.10	98	40		
A6 (Plane 1)	1.00	69	90	72	69
A6 (Plane 2)	0.10	98	10		
A7 (Plane 1)	1.00	76	60	85	76
A7 (Plane 2)	0.10	98	40		
A8 (Plane 1)	0.75	86	50	92	86
A8 (Plane 2)	0.10	98	50		
K5 (Plane 1)	1.00	85	75	88	85
K5 (Plane 2)	0.10	98	25		
K6 (Plane 1)	1.20	80	60	87	80
K6 (Plane 2)	0.10	98	40		
W1 (Plane 1)	0.80	83	60	89	83
W1 (Plane 2)	0.10	98	40		
W2 (Plane 1)	1.00	86	50	92	86
W2 (Plane 2)	0.10	98	50		
W3 (Plane 1)	0.95	82	50	90	82
W3 (Plane 2)	0.10	98	50		

Table 4-15. Kinematic Wave Transform Flow Planes for Urbanized Sub-Basins

Kinematic Wave Collector Channels								
Sub-basin	Length (ft)	Slope (ft/ft)	Manning's n	Area (mi^2)	Shape	Diameter (ft)	Width (ft)	Side Slope (xH:1V)
A1 (Sub-Collector)								
A1 (Collector)	1200	0.015	0.016	0.0207	Circle	3		
A2 (Sub-Collector)								
A2 (Collector)	2500	0.01	0.015	0.03	Circle	4		
A3 (Sub-Collector)								
A3 (Collector)	2800	0.06	0.018	0.03	Circle	3		
A4 (Sub-Collector)								
A4 (Collector)	2200	0.004	0.014	0.03	Circle	4		
A5 (Sub-Collector)								
A5 (Collector)	1200	0.035	0.018	0.03	Circle	2.5		
A6 (Sub-Collector)								
A6 (Collector)	750	0.006	0.06	0.01	Trapezoid		2	10
A7 (Sub-Collector)								
A7 (Collector)	1200	0.035	0.018	0.03	Circle	1.5		
A8 (Sub-Collector)								
A8 (Collector)	2400	0.003	0.015	0.03	Circle	4		
K5 (Sub-Collector)								
K5 (Collector)	1000	0.005	0.016	0.02	Circle	2		
K6 (Sub-Collector)								
K6 (Collector)	2600	0.005	0.018	0.035	Circle	3		
W1 (Sub-Collector)								
W1 (Collector)	1200	0.0015	0.015	0.025	Circle	1.5		
W2 (Sub-Collector)								
W2 (Collector)	800	0.0025	0.015	0.015	Circle	3		
W3 (Sub-Collector)								
W3 (Collector)	900	0.002	0.015	0.015	Circle	3		

Table 4-16. Kinematic Wave Collector Channels

Kinematic Wave Main Channels								
Sub-basin	Route Upstream	Length (ft)	Slope (ft/ft)	Shape	Manning's n	Diameter (ft)	Width (ft)	Slope (xH:1V)
A1	No	1200	0.067	Circle	0.016	3		
A2	Yes	3600	0.001	Trapezoid	0.015		255	0
A3	Yes	800	0.0075	Trapezoid	0.03		50	5
A4	Yes	3100	0.001	Trapezoid	0.035		50	5
A5	No	5800	0.021	Circle	0.015	4		
A6	No	3650	0.001	Trapezoid	0.022		255	0
A7	No	6200	0.0267	Circle	0.015	4		
A8	Yes	2200	0.0015	Trapezoid	0.015		155	0
K5	Yes	700	0.056	Trapezoid	0.035		20	0
K6	Yes	3050	0.049	Trapezoid	0.035		20	0
W1	No	2800	0.0015	Circle	0.016	2		
W2	No	1500	0.0028	Circle	0.014	3		
W3	No	2100	0.0028	Circle	0.015	3		

Table 4-17. Kinematic Wave Main Channels

4.7 Reservoir and Reach Modeling

A number of assumptions were made during hydrologic modeling using the HEC-HMS method. These assumptions were made regarding the reservoir, reach, and junction modeling for the Ala Wai Watershed study area. Building upon the other sub-watershed model calibration, this final model represents the calibration of the entire watershed.

4.7.1 Ala Wai Canal as Reservoir

In order to consider backwater effect caused by the ocean tides, the Ala Wai Canal was modeled as a reservoir by assuming there is an imaginary boundary between the mouth of Canal and the ocean. "A reservoir is an element with one or more inflow and one computed outflow and is modeled by the assumption that water surface in the reservoir is level" (USACE 2008). The routing method was selected as the outflow structure. The size and type of imaginary outlet structure were mainly selected based on the cross section at the mouth of Ala Wai Canal. Noda and Associates (1994) study showed that the channel is a rectangular shape with a dimension of 152 feet x 14 feet near Ala Moana Bridge. The GeoRAS model also created similar cross sections at the mouth of the canal. The inlet elevation for this outlet structure was selected as -6.2 feet which was obtained from the October 30, 2004 storm calibration; then the rise of structure should be about 8 ft. The span of the structure was selected as 152 ft to match the field measurement. Figure 4-17 lists the reservoir model settings and Figure 4-18 shows its related outflow structure. There is no tide gage at the Ala Wai Canal mouth, the tide gage in Honolulu Harbor (NOAA tide level station 1612340) was used to represent the tail water effect. Consequently, the specified stage method was used to represent the main tail water. The elevation-storage function for the reservoir (Ala Wai Canal) was estimated by applying the bathymetric survey data for Ala Wai Canal conducted by Oceanit (2008) and the LiDAR data for surrounding areas, as show in Table 4-18 and Figure 4-18.

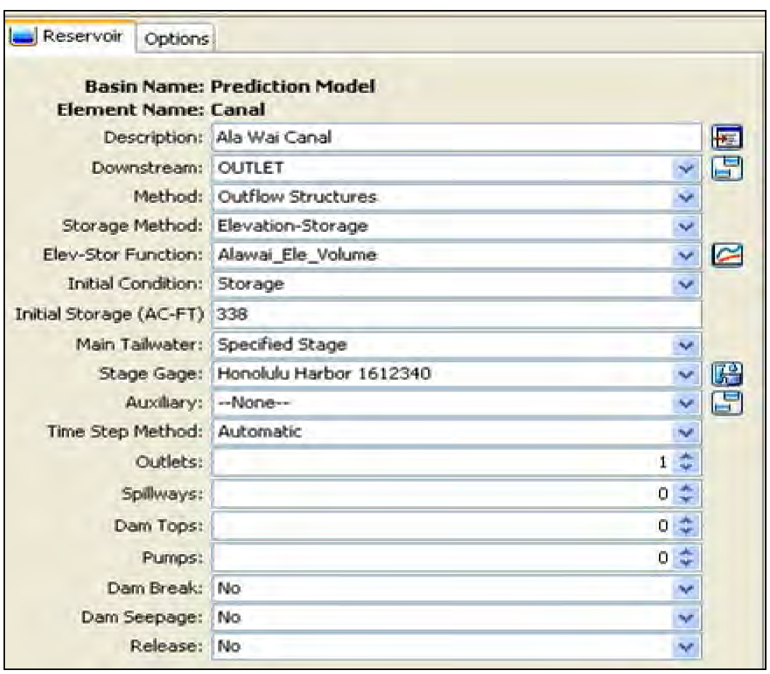

Figure 4-17. Model Settings for Reservoir (Ala Wai Canal)

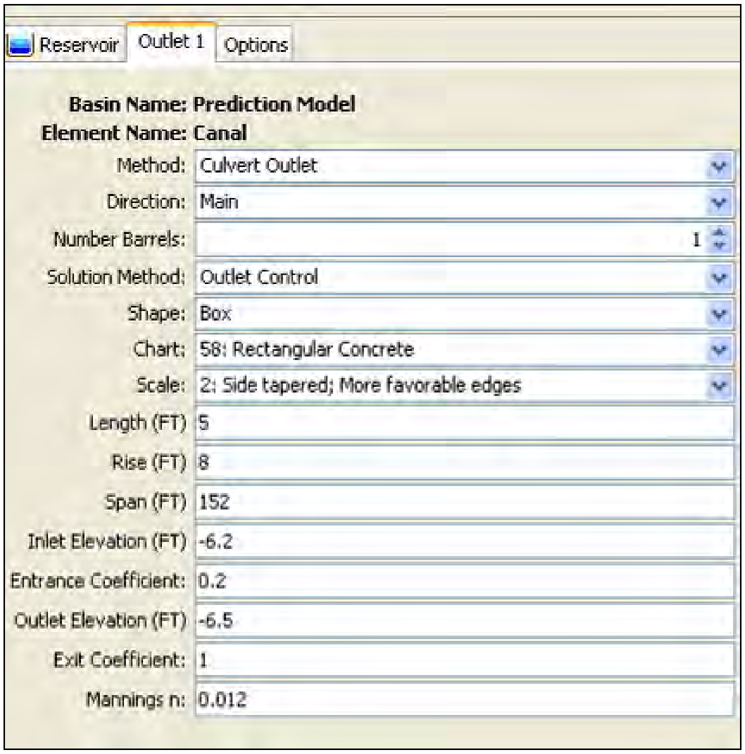

Figure 4-18 Model Settings for the Outflow Structure of Ala Wai Reservoir

Elevation-Storage Function Data

Elevation (ft)	Storage (acre-ft)	Elevation (ft)	Storage (acre-ft)	Elevation (ft)	Storage (acre-ft)
-15.5	0	-7.5	46.21	1	374.51
-15	0.01	-7	58.81	1.5	399.14
-14.5	0.03	-6.5	72.87	2	424.53
-14	0.09	-6	88.32	2.5	451.72
-13.5	0.22	-5.5	105.22	3	481.35
-13	0.43	-5	123.38	3.5	516.12
-12.5	0.74	-4.5	142.62	4	565.43
-12	1.2	-4	162.89	4.5	649.36
-11.5	1.89	-3.5	183.98	5	790.16
-11	2.99	-3	205.54	5.5	994.63
-10.5	4.89	-2.5	227.5	6	1260.41
-10	8.01	-2	249.61	6.5	1576.57
-9.5	12.49	-1.5	271.72	7	1930.93
-9	18.44	-1	293.83	7.5	2313.63
-8.5	25.97	-0.5	315.94	8	2718.57
-8	35.25	0	338.05		
-7.5	46.21	0.5	350.41		

Table 4-18. Elevation-Storage Curve Function Data

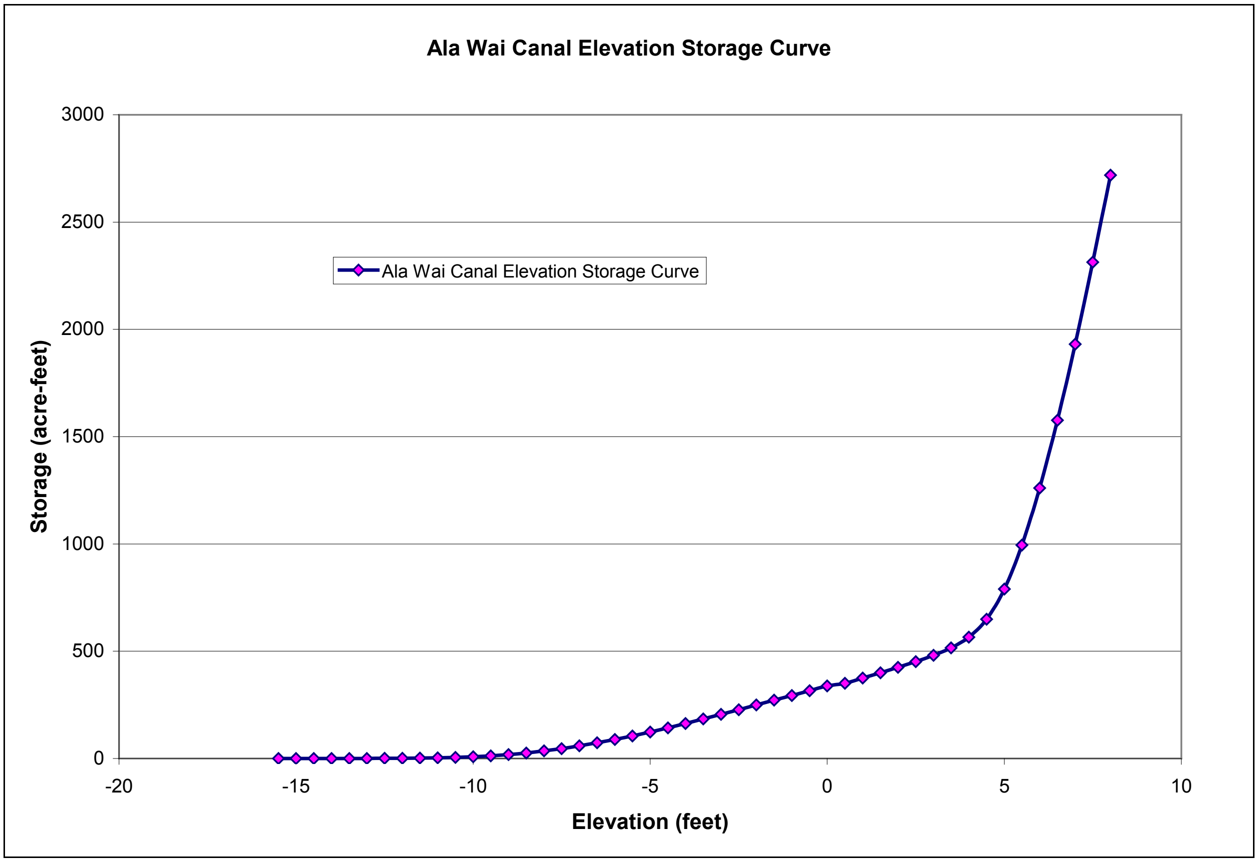

Figure 4-19. Elevation Storage Curve for Ala Wai Canal

The reservoir model was calibrated using the observed stage in the Ala Wai Canal from the October 2004 and December 1967 storm events. For modeling of the 2004 storm, the recorded stream flow hydrograph at USGS stream gage 16247100 was used to represent the inflow from upstream of Manoa and Palolo Streams; and Makiki calibrated model hydrograph at JK2 (USGS stream gage 16238000) was used to represent the inflow from Makiki area. Figure 4-20 illustrates the HEC-HMS model layout for October 30, 2004 storm calibration. Figure 4-21 shows the modeled and observed stages in Ala Wai Canal. The stage peak time matched very well at about 20:15pm with only 0.1 foot difference between the observed stage and the modeled stage.

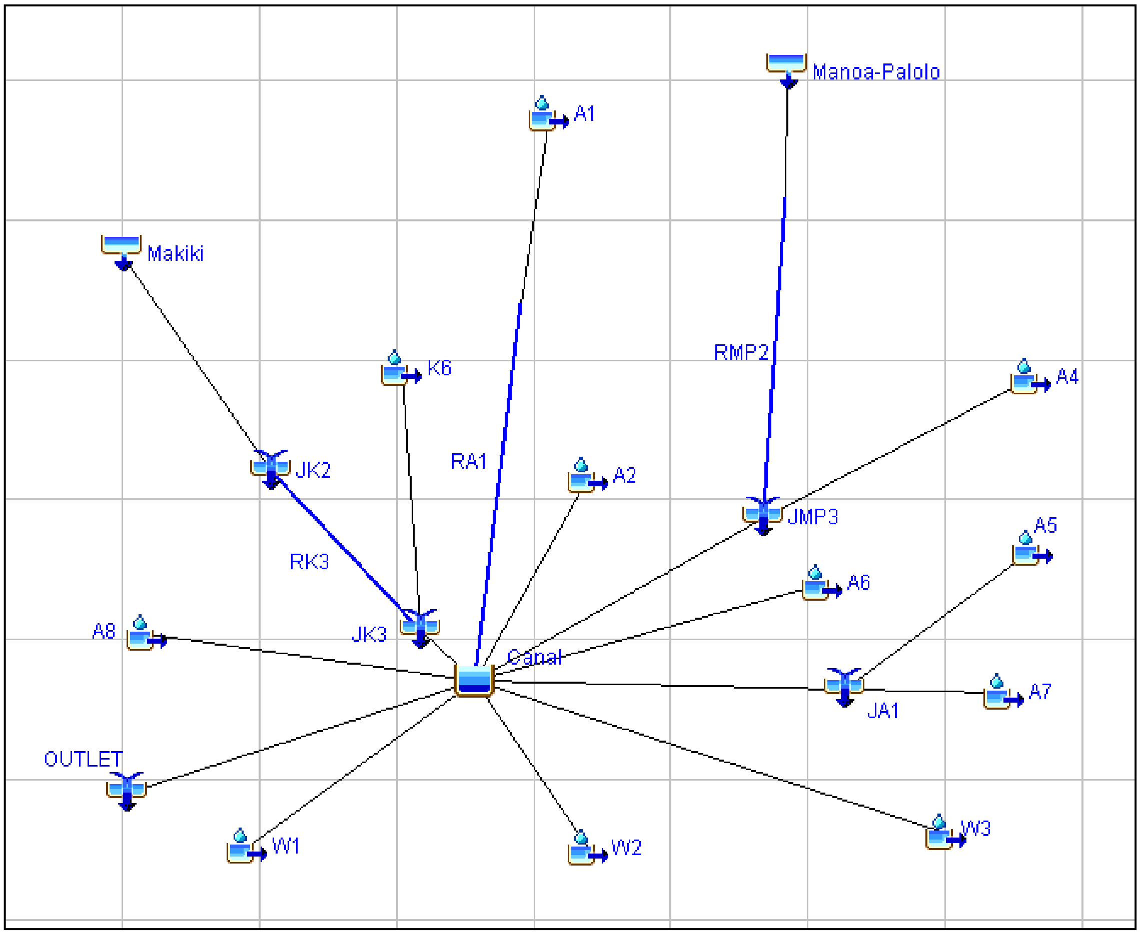

Figure 4-20. Model Layout for 2004

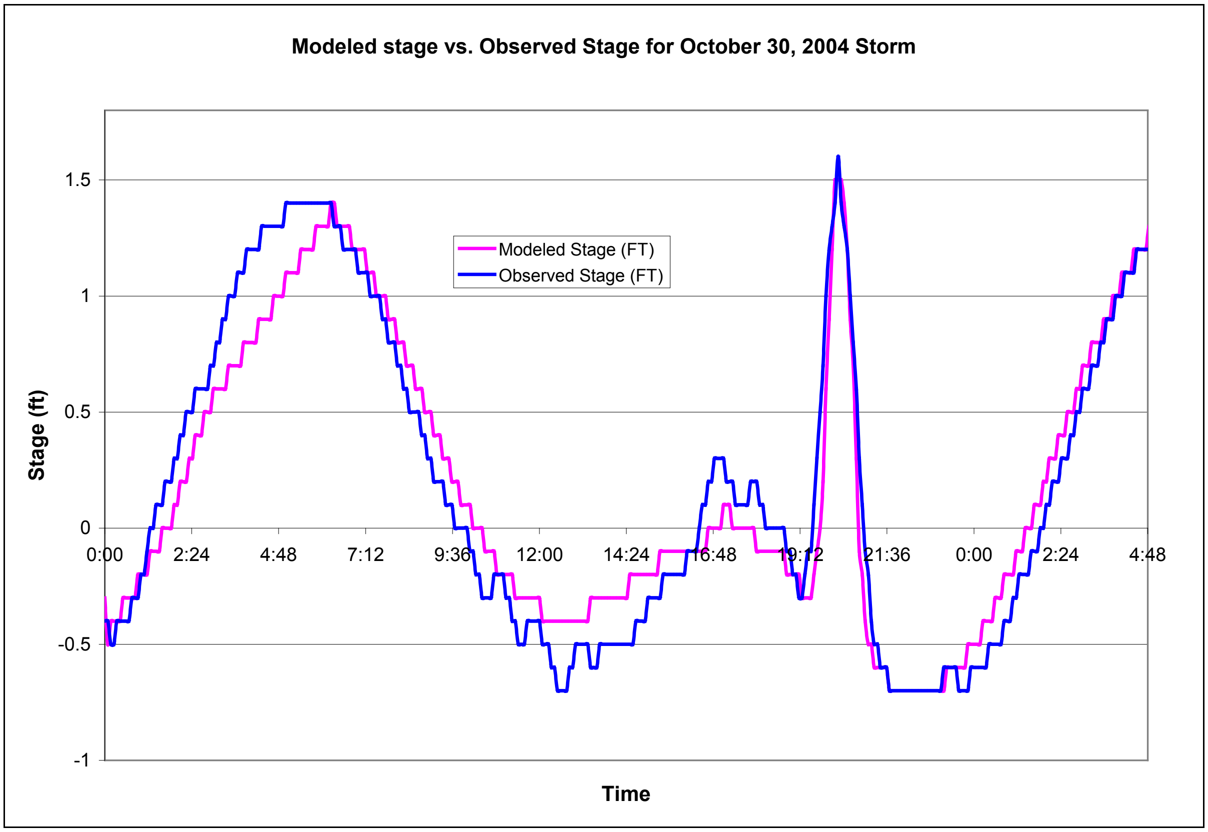

Figure 4-21. Calibrated Water Elevation vs. Observed Stage for Ala Wai Canal on October 30, 2004 Storm

For modeling of the December 17—18, 1967 storm, the calibrated Mānoa-Pālolo model hydrograph at USGS stream gage 16247100 was used to represent the upstream inflow. The finalized Makiki model described in Section 4.5 was used to represent the Makiki sub-watershed. Figure 4-22 shows the HEC-HMS model layout for calibrating this storm. The DLNR post flood report (1968) noted that Ala Wai Canal in Waikiki overflowed at the confluence with Mānoa-Pālolo Drainage Canal. Ala Wai Boulevard and adjacent streets near the confluence were flooded with water up to two feet deep (DLNR, 1968). The modeled peak stage was about 4.4 feet, or about 2.2 feet above Ala Wai Boulevard. Figure 4-23 shows the modeled stage in feet.

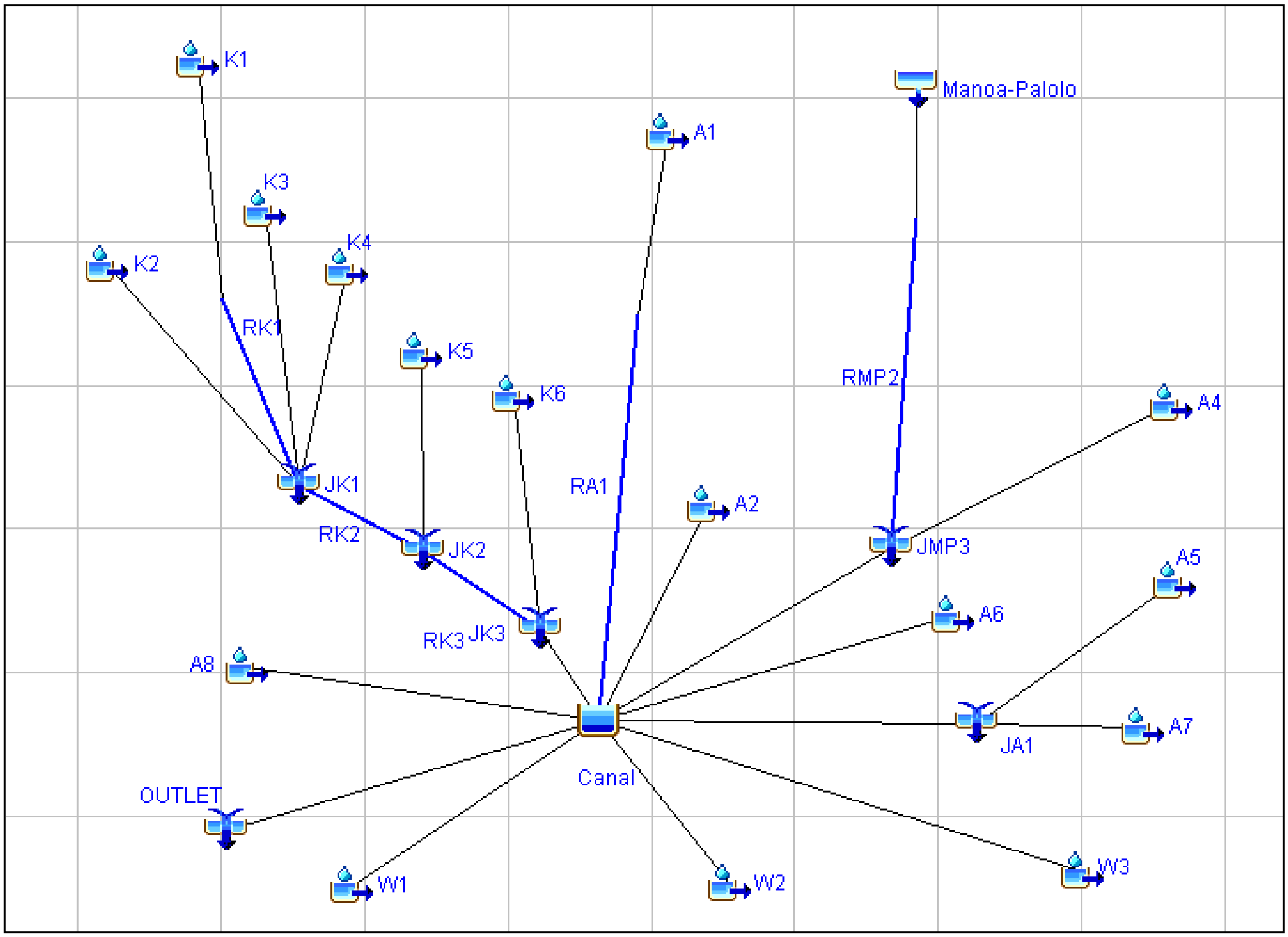

Figure 4-22. Model Layout for 1967

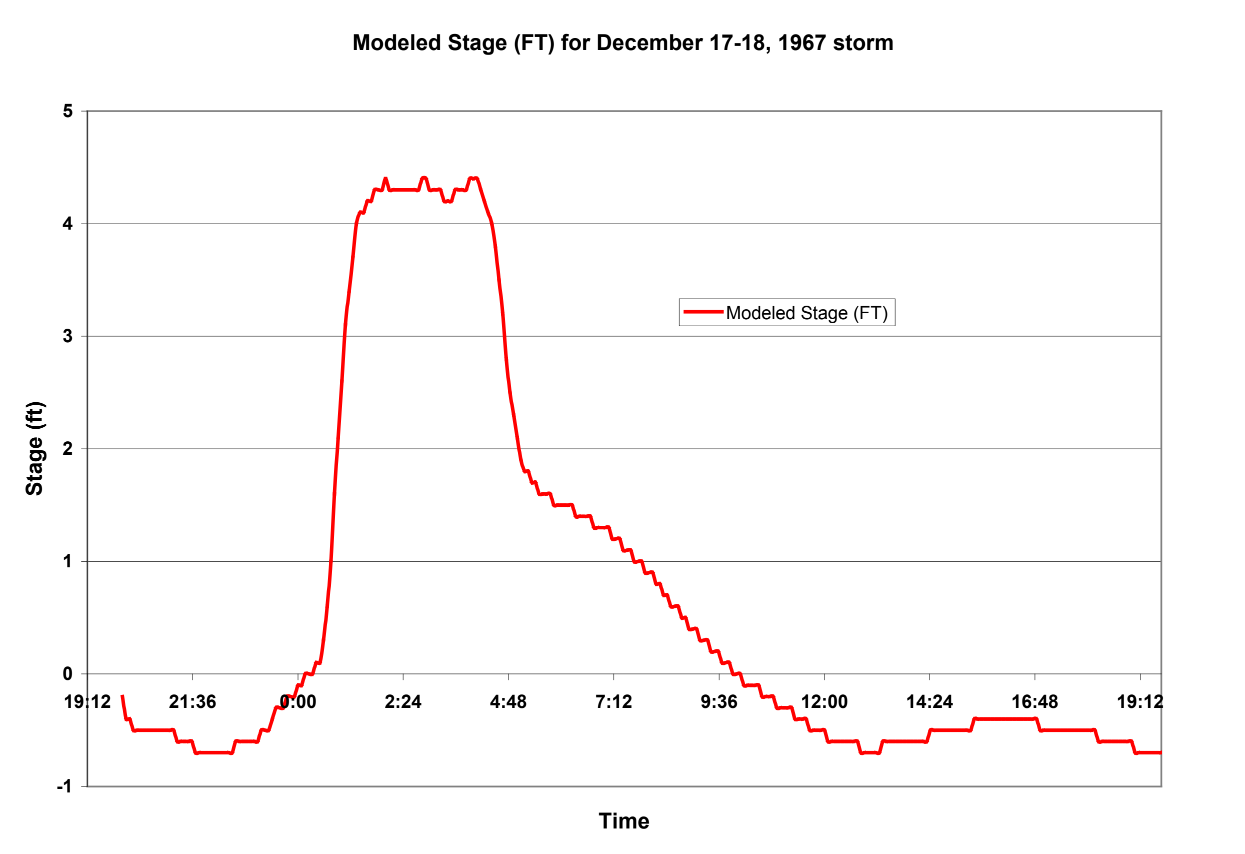

Figure 4-23. Calibrated Water Elevation at Ala Wai Canal for December 17-18, 1967 Storm

4.7.2 Reaches: Muskingum-Cunge and Modified Puls Channel Routing

The Muskingum-Cunge channel routing parameters were used and included the Manning's *n* values, length, slope, and cross-sections. The Manning's *n* values for the stream channel and its banks were determined using Chow's (1959) guidelines and channel conditions. The length of each reach was determined using GIS Arcview 3.3 maps; the slopes were estimated using contours generated from LiDAR data; and the widths were determined from field measurements and the cross-sectional data obtained from GeoRAS. The channel routing parameters are shown in Table 4-19.

Muskingum-Cunge Channel Routing for HEC-HMS Model

Reach	Length (ft)	Slope (ft)	Manning's n	Shape	Width (ft)	Side Slope (xH:V)
RK1	4350	0.0415	0.05	Trapezoid	10	2
RK2	2650	0.0101	0.03	Trapezoid	20	2
RM7	1180	0.008	0.035	Trapezoid	50	2
RMP1	1900	0.0053	0.04	Trapezoid	50	2
RP1	5900	0.056	0.046	Trapezoid	15	2
RP2	3300	0.015	0.04	Trapezoid	15	2
RP3	4350	0.04	0.04	Trapezoid	12	2
RP4	5950	0.0185	0.0162	Rectangle	30	
RP5	4300	0.0186	0.0162	Rectangle	30	

Table 4-19. Muskingum-Cunge Channel Routing for HEC-HMS Model

The Modified Puls Routing Method was used for the Ala Wai Canal modeling to take backwater effects into consideration. The Modified Puls Routing Method is also called storage routing or level pool routing and is most often applied to reservoir routing. Because the Ala Wai Canal was modeled as a reservoir, the stream reaches that discharge into the reservoir were modeled using the Modified Puls Routing Method. The storage-discharge functions for reaches RMP2 (Mānoa-Pālolo Canal) and RK3 (Makiki Stream) were defined based on the elevation-discharge measurements of stream gages 16247100 at the Mānoa-Pālolo Canal and 16238000 at King Street bridge. The storage-discharge function for reach RA1 (Alanaio Stream) was defined by using Manning's equation. Figure 4-24 shows the locations of these three reaches. Figures 4-25, 4-26, and 4-27 Show the storage-discharge curves for these three reaches, respectively.

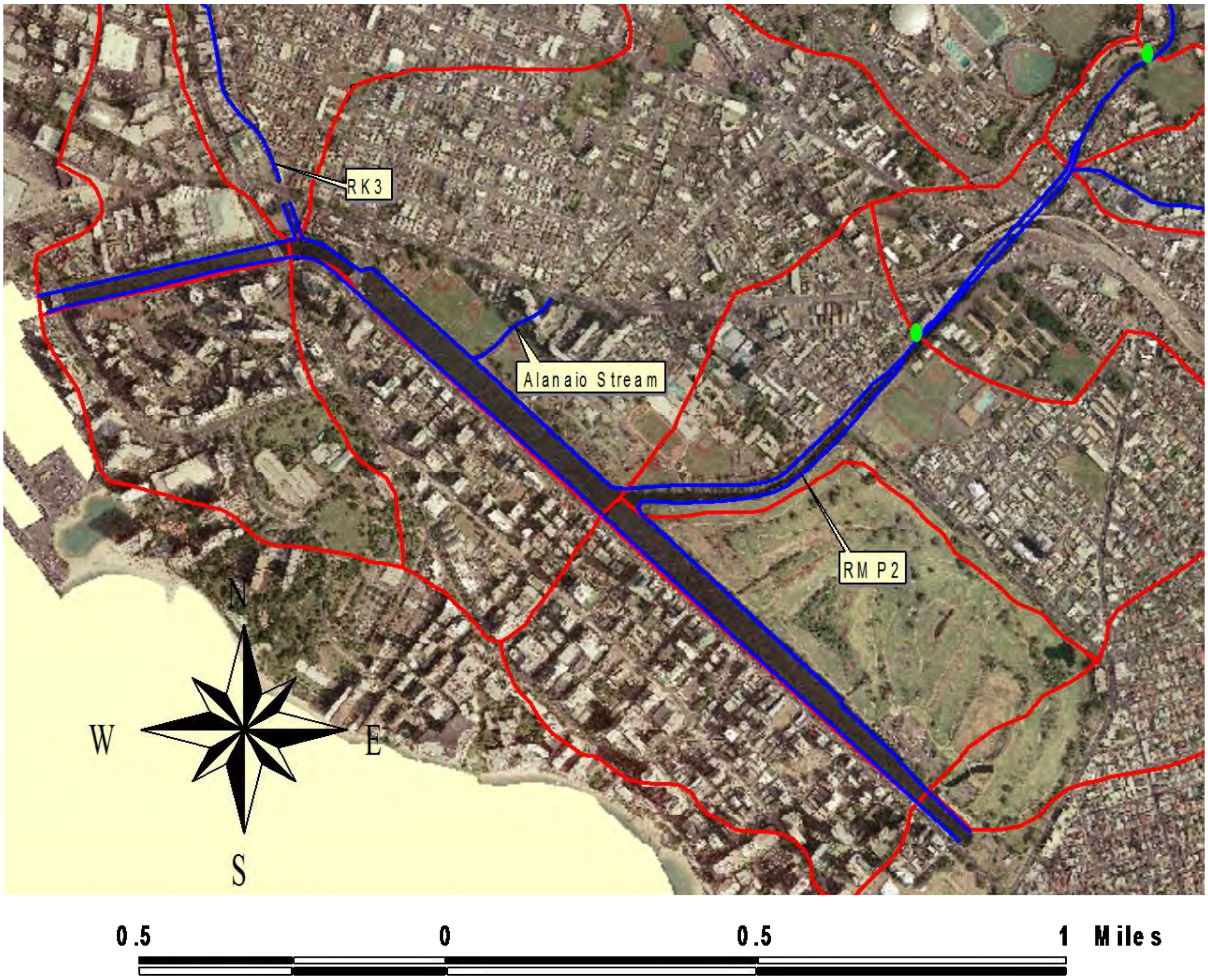

Figure 4-24. Reach locations for Modified Puls Routing Method

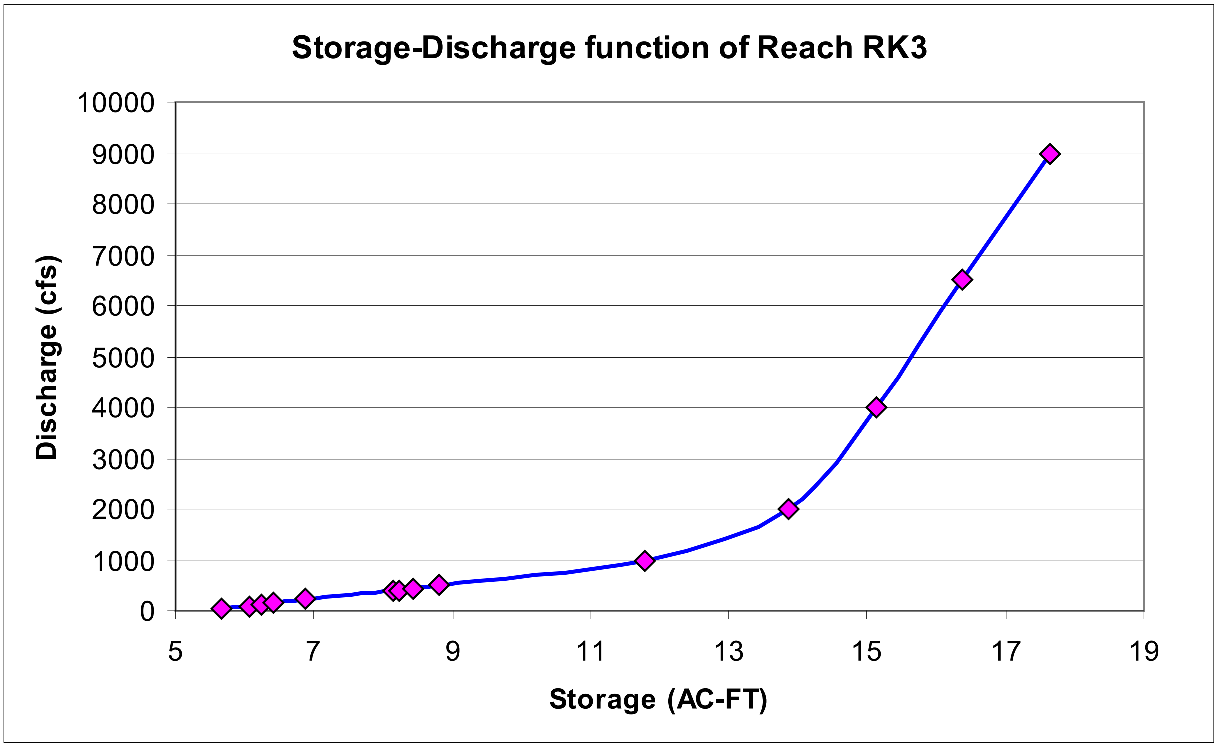

Figure 4-25. Storage-Discharge Curve for Reach RK3

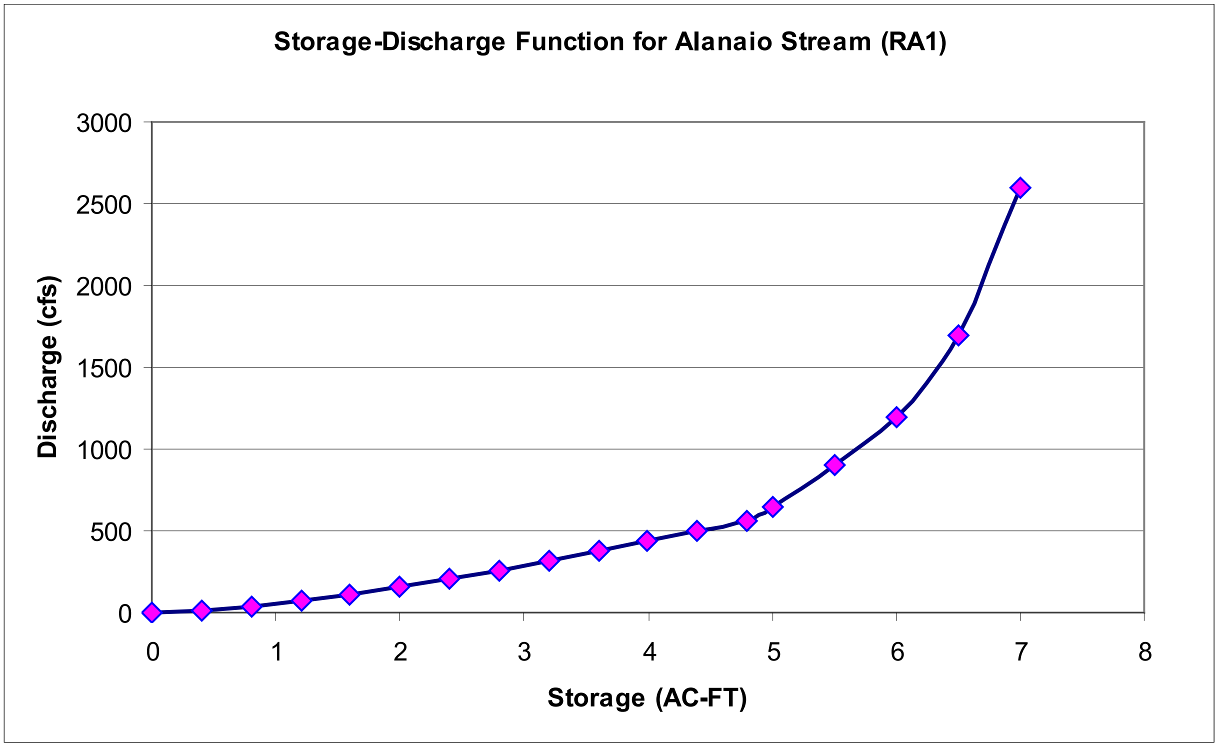

Figure 4-26. Storage-Discharge for Reach RA1

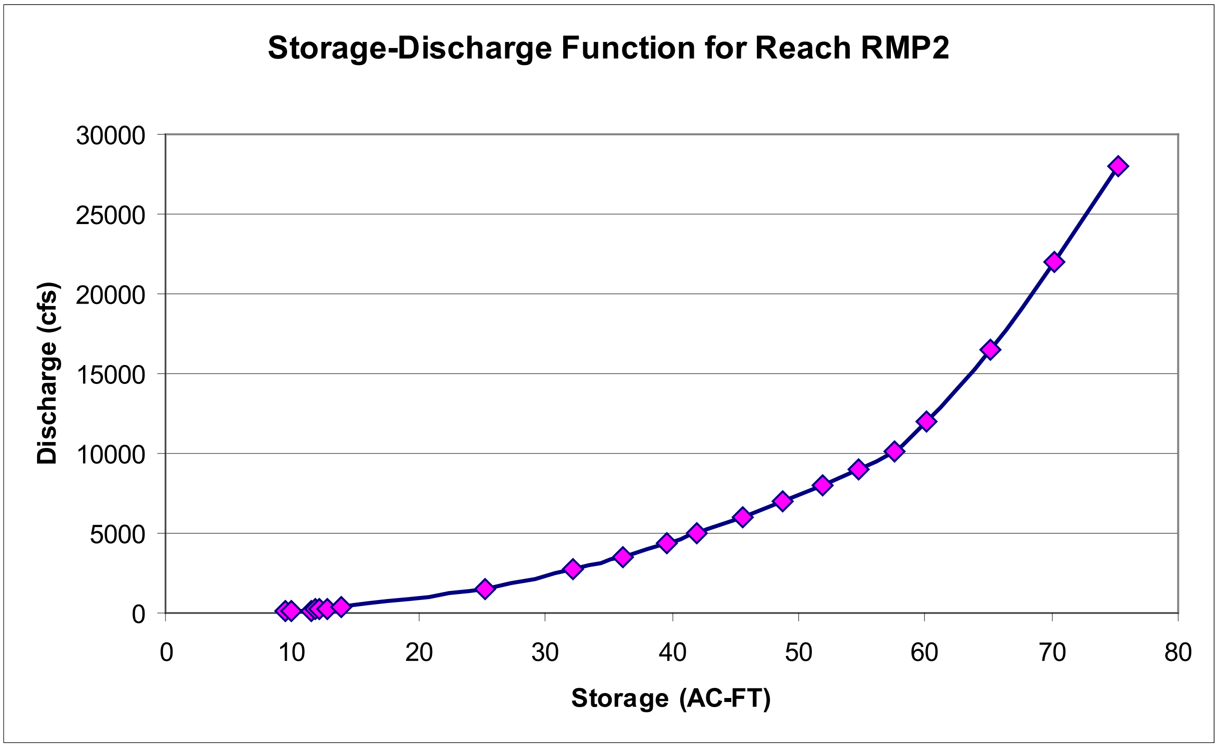

Figure 4-27. Storage-Discharge for Reach RMP2

4.8 Inflow Hydrographs at Kānewai Gage

For consistency with the previous Mānoa Watershed Project hydrologic study, the final results from that study were used to represent the whole Mānoa sub-watershed at the Kānewai Field stream gage. Inflow hydrographs were obtained from the HEC-HMS model of the Mānoa Watershed Project study for the storm chance exceedances of 50 through 0.2 percent. Table 4-20 lists the peak discharges at the Kānewai Field stream gage (USGS 16242500). Figures 4-28 and 4-29 provide the modeled stream flow at Kānewai Field, based on the results from the Mānoa Watershed Project hydrologic study (Oceanit 2008).

Peak Discharges at Kānewai Field Stream Gage from Mānoa Watershed Project								
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceeded	50%	20%	10 %	5%	2%	1%	0.5 %	0.2 %
Peak Discharges (cfs)	2,500	4,300	6,000	7,600	9,500	10,700	12,000	14,000

Table 4-20. Peak discharges at Kānewai Field Stream Gage from Mānoa Watershed Project Hydrologic Study

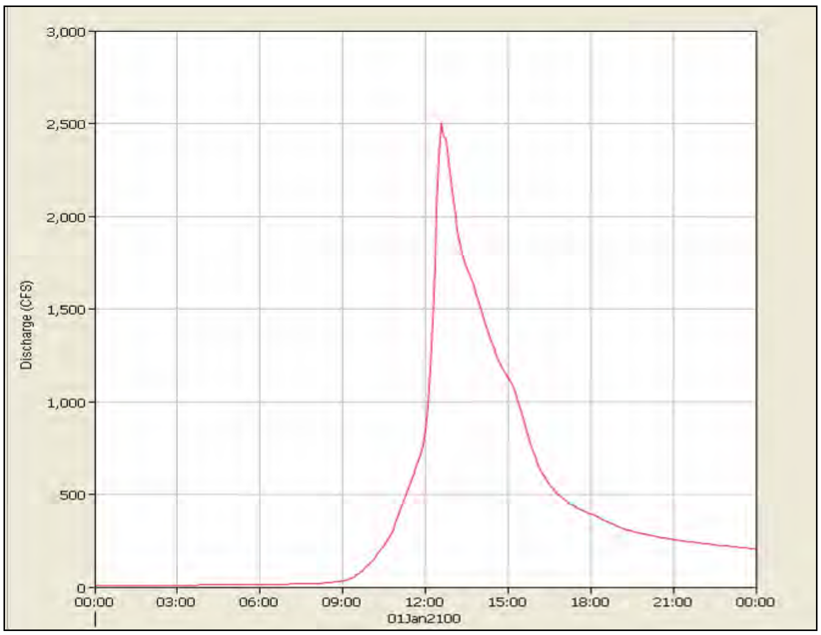

Figure 4-28. Inflow Hydrograph for the 50-percent Chance Flood Used to Represent the Manoa Sub-Watershed in the Ala Wai Watershed HEC-HMS Model (at Kānewai Field)

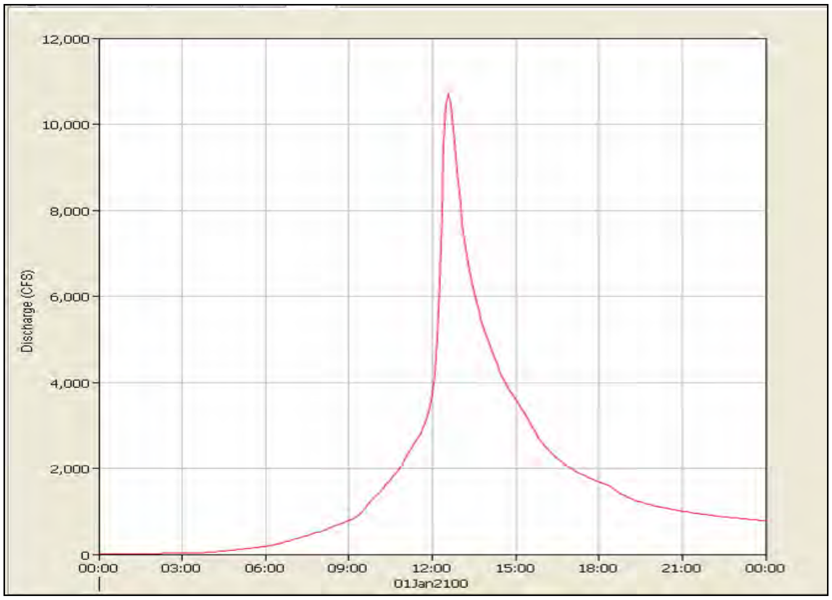

Figure 4-29. Inflow Hydrograph for the 1-percent Chance Flood Used to Represent the Manoa Sub-Watershed in the Ala Wai Watershed HEC-HMS Model (at Kānewai Field)

4.9 Peak Flow Results

For predicting the peak discharges for various return periods, the frequency storm with an intensity position at 50 percent was used in computing the peaks and hydrographs. The HEC-HMS model predicted peak discharges at various junctions in the Ala Wai Watershed are listed in Table 4-21. The final HEC-HMS model layout is shown below in Figure 4-30.

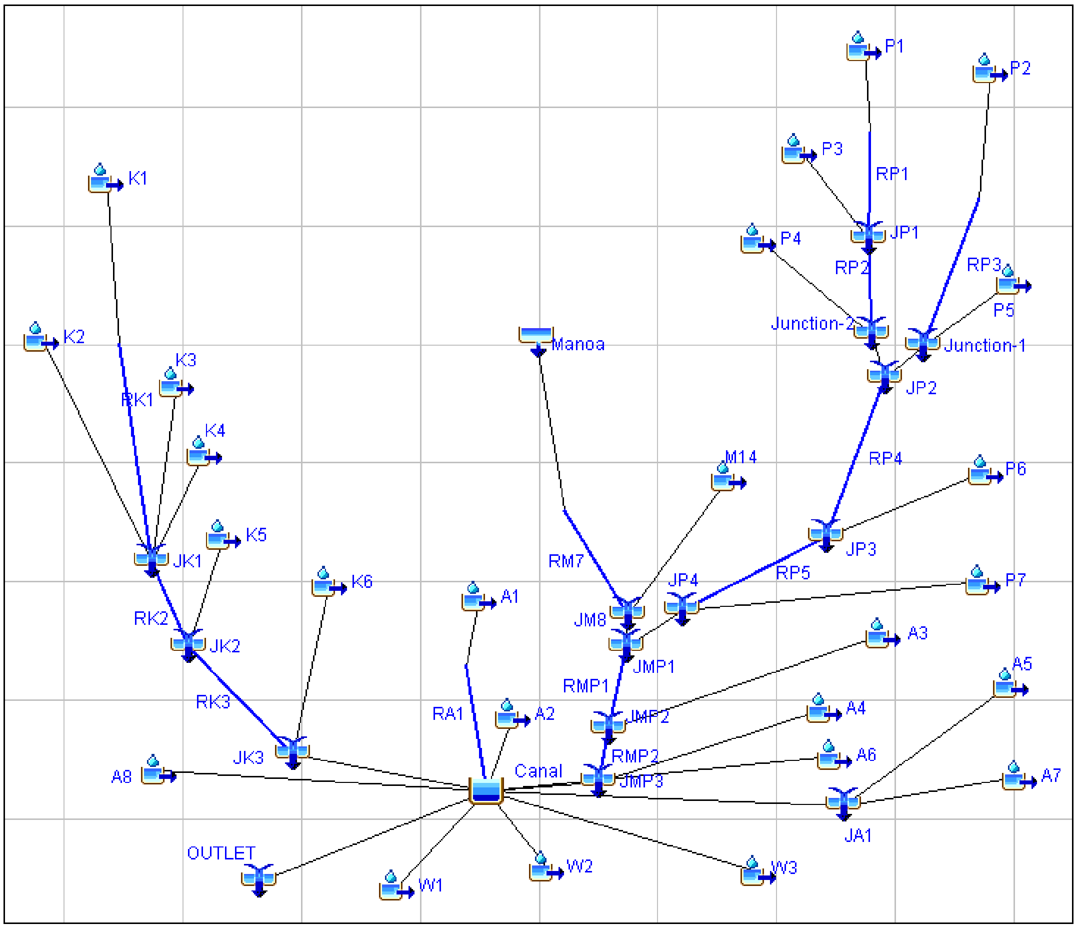

Figure 4-30. Ala Wai Watershed HEC-HMS Model

HEC-HMS Model Results—Peak Flow Discharges at Junctions

Table 4-20 HEC-HMS Model Peak Flow Discharges at Junctions								
	Peak flow discharge (cfs)							
Return Period (yr)	2	5	10	20	50	100	200	500
Percent Chance Exceedance	50%	20%	10%	5%	2%	1%	0.5%	0.2%
JK1	570	1,200	1,890	2,400	3,150	3,740	4,380	5,240
JK2	660	1,360	2,110	2,650	3,440	4,060	4,730	5,630
JK3	890	1,770	2,690	3,340	4,280	5,000	5,790	6,850
JM8	2,560	4,450	6,210	7,860	9,810	11,100	12,400	14,500
JP1	320	730	1,150	1,460	1,900	2,220	2,590	3,110
JP2	940	2,030	3,190	4,010	5,180	6,040	6,980	8,320
JP3	1,330	2,710	4,170	5,180	6,620	7,670	8,850	10,500
JP4	1,550	3,120	4,720	5,810	7,400	8,550	9,860	11,600
JMP1	4,020	7,170	10,300	12,900	16,100	18,500	20,900	24,400
JMP2	4,090	7,340	10,500	13,000	16,300	18,700	21,100	24,700
JMP3	4,220	7,450	10,700	13,300	16,600	18,900	21,400	24,900
Ala Wai Canal	6,000	10,100	13,400	15,200	16,700	17,700	18,700	20,500

Table 4-21. HEC-HMS Model Predicted Peak Discharges at Junctions

4.10 USGS Regression Equations and City and County's Plate 6

Regional regression equations developed by the USGS (Wong, 1994) for estimating peak discharges for the 50-, 20- 10-, 4-, 2-, and 1-percent chance exceedance probabilities at gaged and ungaged sites were used to calculate peak flows in the sub-watersheds. The equations for Leeward O'ahu were used for the sub-watersheds in this study. The drainage area (DA) and median annual rainfall (P) in these equations are independent parameters. These regression equations are valid for ungaged sites when (1) the drainage areas are between 0.03 and 45.7 square miles; (2) where less than 36 percent of the area is urbanized; and (3) the median rainfall is between 29 and 239 inches. The median annual rainfall for each sub-watershed was determined from DLNR (1982). The median annual rainfall amounts for the junctions were calculated by the weighting mean method with respect to the sub-watershed areas. The equations used bias-correction factors along with the accuracy of the estimates in equivalent years of record (Wong, 1994). The peak discharges calculated using these regression equations and Plate 6 of the City's drainage standards (2000) for each junction are presented in Table 4-22. The accuracy of these results is 16 years for the 1 percent chance exceedance event and 15 years for the other storm events.

The City storm drainage standards (2000) specify the use of the rational method for drainage areas of 100 acres or less and Plate 6 for drainage areas greater than 100 acres, and this method was used for some of the sub-basins in the Ala Wai Watershed study area. Plate 6 is an envelope curve developed from maximum known peaks and regression analysis of 100-year peak flows. This curve is assumed to represent a 100-year peak flow but actually has a slightly higher return period (Wong 1994). The accuracy of this curve is based not on the average years of recorded data but by the standard error of regression. The accuracy of data used for peak determination of the 100-year envelope is unknown. In the absence of accurate data, an equivalent years of record of 10 years is assigned (Interagency Advisory Committee on Water Data 1982).

Plate 6 was applied to calculate the 100-year peak discharges in all sub-basins because the corresponding drainage areas exceed 100 acres. Plate 6 provides three curves relating to the peak discharge of the 100-year return period storm (1 percent chance exceedance probability). Curve B from Plate 6 was used for the Mānoa sub-watershed.

USGS Regression Equations and Plate 6 Calculation in cfs

RP	Equation with BFC	JK1	JK2	JM8	JP1	JP2	JP3	JP4	JMP1	JMP2	Years of Record
2	Q_2=3.635 $(DA)^{0.634}$ $P^{1.08}$	660	670	1660	650	1040	1040	1040	2120	2110	**4.2**
5	Q_5=27.58 $(DA)^{0.642}$ $P^{0.773}$	1340	1370	3100	1160	1930	2020	2060	4060	4080	**5.8**
10	Q_{10}=77.32 $(DA)^{0.646}$ $P^{0.621}$	1960	2000	4330	1580	2700	2870	2970	5760	5800	**8.2**
25	Q_{25}=225.7 $(DA)^{0.646}$ $P^{0.464}$	2900	2980	6120	2200	3830	4150	4330	8240	8320	**11.4**
50	Q_{50}=440.7 $(DA)^{0.645}$ $P^{0.368}$	3840	3960	7870	2810	4940	5410	5690	10680	10810	**13.7**
100	Q_{100}=788.3 $(DA)^{0.643}$ $P^{0.286}$	4680	4840	9330	3320	5880	6500	6860	12740	12910	**15.8**
	Plate 6 (100-yr)	5300	5600	11000	3200	6500	7700	8100	15500	16000	**10**

Table 4-22. USGS Regression Equations and Plate 6 Calculation

4.11 FEMA Flood Insurance Study

The hydrologic and hydraulic analysis for the original Flood Insurance Study (FIS) for the City and County of Honolulu was performed by R.M. Towill Corporation in 1976. FEMA revised the previous FIS for the City and published the most updated FIS in 1979.

For Makiki Stream, USGS regression equations were used to obtain peak flow discharges for the 10-, 50-, and 100-year flooding events (FEMA, 2004). The 500-year flood was determined by a regression equation utilizing the same basic data and regression techniques as applied by USGS. These regression equations applied the ratio of the drainage area covered by forests and vegetation to total drainage area in percent instead of the median rainfall that current USGS regression equations applied to determine the peaks. Figure 18 in FIS (FEMA, 2004) was the results that only applied to one place with the drainage area as about 2.49 square miles. This drainage area is equal to the drainage area of junction JK2; in other words, only junction JK2 is available to have FEMA flood insurance analysis peak flow discharges.

For Palolo Stream, peak discharges were based on a statistical analysis results by using the 25-year recording annual peaks at USGS Gaging Station 16247000. The analysis followed the standard log-Pearson type III method procedures as outlined by the Water Resources Council. So the FEMA FIS analysis for Palolo Stream is only applied to junction JP3 that USGS gage 16247000 located.

For Manoa-Palolo and Ala Wai Canals, the peak discharges were determined by using SCS hydrograph method. Probably because of the higher proportion of urbanized areas, the SCS method resulted in slightly higher peak discharges.

For JM8, which is part of Manoa sub-watershed, same analysis was used as previous Manoa watershed study conducted by Oceanit (2008).

Table 4-23 shows the FEMA flood insurance study analysis for Makiki, Palolo, Manoa-Palolo Canal and Ala Wai Canal.

Peak Flow Discharges in cfs Calculated by FEMA				
Return Period (yr)	10	50	100	500
Percent Chance Exceedance	10%	2%	1%	0.2%
JK2	1,850	3,250	3,950	5,950
JM8	7,600	11,500	13,600	17,000
JP3	2,790	4,510	5,340	7,530
JMP2	12,000	19,200	23,000	28,500
Ala Wai Canal	13,700	23,000	28,200	36,200

Table 4-23. Peak Flow Discharges Calculated by FEMA

4.12 Flow Frequency Analysis

HEC-SSP version 1.0 Beta was used to perform the flow frequency analysis. This software is limited to performing flood flow frequency analysis based on Bulletin 17B, "Guidelines for Determining Flood Flow Frequency" (Interagency Advisory Committee on Water Data 1982). Three USGS stream gages that have sufficient data to perform the flow frequency analysis are within the study area. The USGS stream gage 16247100 at the Mānoa-Pālolo Canal (Junction JMP2), adjacent to the Kaimukī High School, has a drainage area of 10.34 square miles. Thirty-eight effective annual peaks were used in the HEC-SSP model to predict the peaks for the various return periods at this junction. The USGS Pālolo Stream gage 16247000 (Junction JP3) has a drainage area of 3.62 square miles. Thirty-two effective annual peaks were used at this junction. The USGS Pūkele Stream (tributary of Pālolo Stream) gage 16244000 (Junction JP1) has a drainage area of 1.15 square miles. Fifty-nine effective annual peaks were used at this junction. The following figures and tables show the flow frequency results from HEC-SSP model (Figures 4-31-33 and Tables 4-24 through 4-26).

At USGS Gaging Station 16247000, there are 32 effective annual peaks available to perform the statistical frequency analysis. The continuous recorded annual peaks are from 1953 to 1979 and from 2003 to 2007, but no data is available between 1980 and 2002. The recorded annual peaks from 2003 to 2007 seem incorrect for the following two reasons.

(1) On October 30, 2004, the recorded peak at this gage was 776 cfs. The tributary stream gage upstream (Pukele) recorded a 753 cfs peak, and another tributary (Waiomao Stream) received the same rain as Pukele Stream received. At USGS gage 16247100 downstream, the recorded peak was 9380 cfs and the Manoa Stream at Kanewai gage recorded a peak at 5860 cfs. Thus, the peak flow at the Palolo gage should be in a range of 1500 to 3000 cfs rather than the 776 recorded because it received similar rainfall as Manoa.

(2) The peak for March 31, 2006 storm at Palolo Stream Gage was 1390 cfs, at downstream gage USGS 16247100, the recorded peak was 9320 cfs, the rainfall was uniformly distributed into the study area, the Palolo valley should have generated a range 2000 to 3000 cfs peak flow. Since there was possible channel conditions changed during the last 50 years, the data in this gage may be lower than actual stream flows, as a result, the HEC-SSP and FEMA analysis (used 25-year annual peaks) got lower peak discharges.

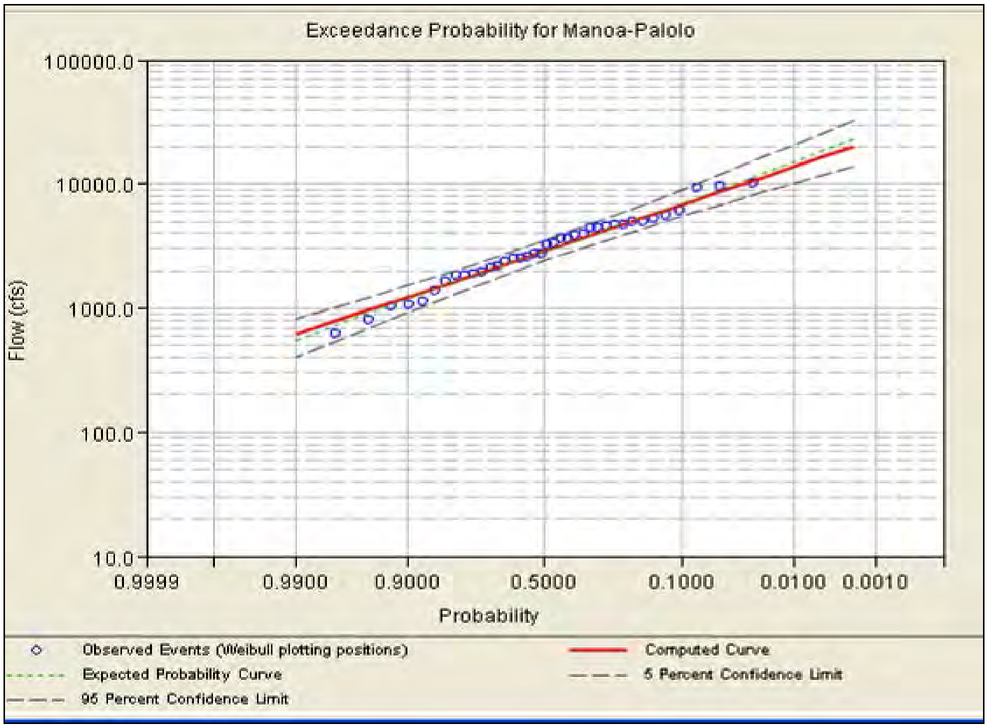

Figure 4-31. Exceedance Probability for Mānoa-Pālolo Canal Stream Gage JMP2 (USGS Stream Gage[16247100])

Percent Chance Exceedance	Return Period (year)	Computed Flow (cfs)	Confidence Limits Flow (cfs)	
			0.05	0.95
0.2	500	19,800	32,538	13,949
0.5	200	16,200	25,443	11,719
1	100	13,700	20,783	10,143
2	50	11,400	16,677	8,654
5	20	8,670	12,017	6,804
10	10	6,800	9,013	5,475
20	5	5,070	6,407	4,179
50	2	2,880	3,459	2,404

Table 4-24. Flood Flow Frequency Results for Mānoa-Pālolo Canal Stream Gage JMP2 (USGS Stream Gage [16247100])

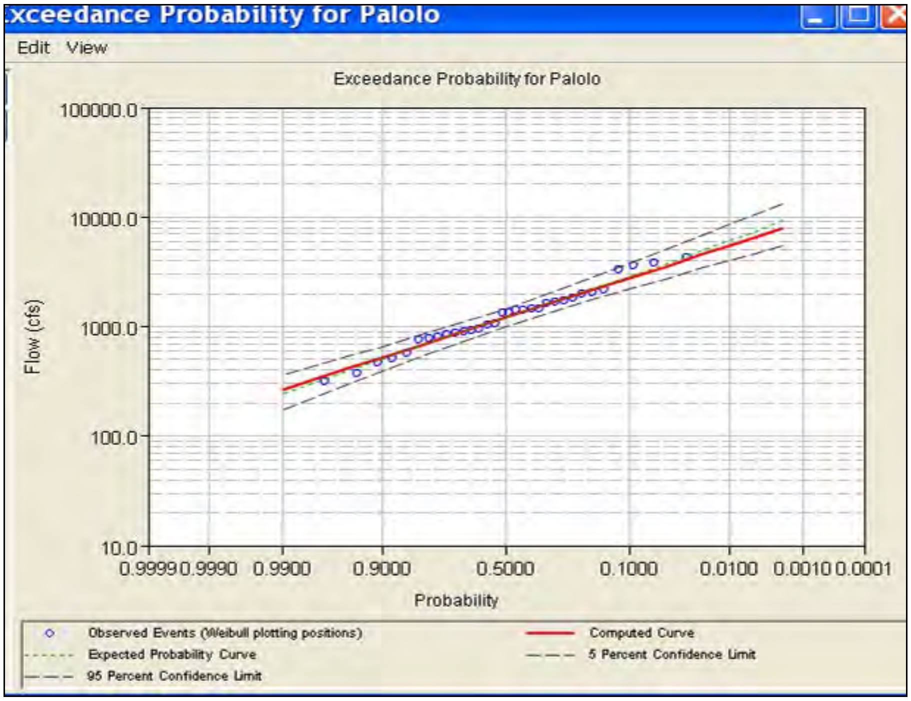

Figure 4-32. Exceedance Probability for Pālolo Stream Gage JP3 (USGS Pālolo Gage [16247000])

Percent Chance Exceedance	Return Period (year)	Computed Flow (cfs)	Confidence Limits Flow (cfs)	
			0.05	0.95
0.2	500	7,820	13,366	5,422
0.5	200	6,430	10,478	4,589
1	100	5,470	8,578	3,996
2	50	4,580	6,900	3,433
5	20	3,510	4,991	2,725
10	10	2,780	3,757	2,212
20	5	2,090	2,683	1,705
50	2	1,210	1,466	997

Table 4-25. Flood Flow Frequency Results for Pālolo Stream Gage JP3 (USGS Pālolo Gage [16247000])

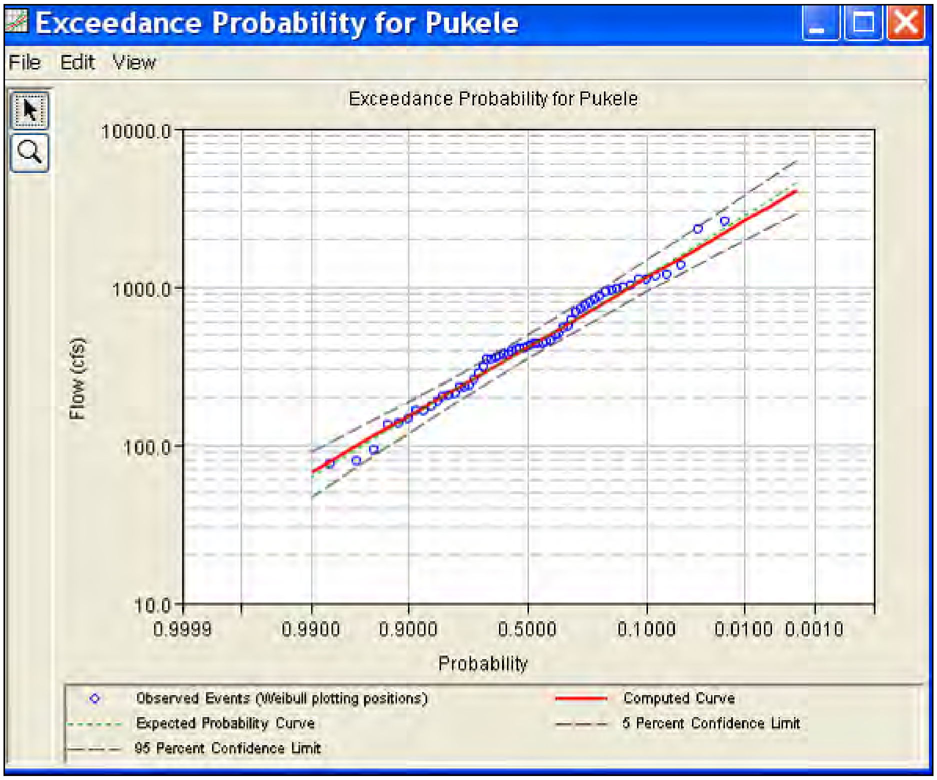

Figure 4-33. Exceedance Probability for Pūkele Stream Gage JP1 (USGS Pūkele Gage [16244000])

Percent Chance Exceedance	Return Period (year)	Computed Flow (cfs)	Confidence Limits Flow (cfs)	
			0.05	0.95
0.2	500	4,050	6,330	2,880
0.5	200	3,190	4,800	2,330
1	100	2,620	3,820	1,960
2	50	2,110	2,980	1,620
5	20	1,530	2,060	1,210
10	10	1,150	1,490	930
20	5	810	1,010	680
50	2	420	500	350

Table 4-26. Flood Flow Frequency Results for Pūkele Stream JP1 (USGS Pūkele Gage [16244000])

5 Results of Hydrologic Model

All of the hydrologic analysis methodologies estimate peak flow discharges (cfs) for return periods (percent chance exceedance storms) by junction; the methodologies include the HEC-HMS modeling, the USGS regression method, City Plate 6, FEMA Flood Insurance Study, and the HEC-SSP model. Each of these methodologies provides a predictive measure for peak discharges, and used together they offer a clear and accurate depiction of where peak flows will occur during 50, 20, 10, 5, 2, 1, 0.5, and 0.2 percent chance exceedance storms.

5.1 Determination of Final Peak Flow Discharges

The USACE Engineer Manual (EM) 1110 – 2 -1619 (1996, Table 4-5, page 4-5) provides guidelines to assign accuracies to flood frequency estimates determined by various methods in term of equivalent years of record. It is assumed that the estimates with higher equivalent years of record are more reliable than those with lower equivalent years of record. Based on the guidelines, the HEC-SSP model is the most reliable with equivalent years of record 59, 32, and 38 for junctions JP1, JP3, and JMP2, respectively. The HEC-HMS model was calibrated to three historical storms for Manoa and Palolo sub-watersheds, two historical storms for Ala Wai Canal reservoir model, and one historical storm event for Makiki sub-watershed. Although there was no calibration to the urbanized sub-basins, the parameters physical measurable Kinematic Wave transform method was applied. An equivalent record length of 20 years was assigned to the results generated by HEC-HMS model based on guidelines provided in EM110-2-1619 (USACE, 1996).

FEMA flood insurance study within Ala Wai watershed area applied various methods to determine the peak discharges, based on the analysis done with equivalent record lengths of 15 years and were assigned to FEMA results in junctions JK2, JM8, JMP2, and Ala Wai Canal. An equivalent record length of 25 years was assigned to FEMA results in junction JP3 in response to its statistic analysis using 25-year recorded annual peaks. The weighting factors for the HEC-HMS modeling, the USGS regression, City Plate 6, FEMA Flood Insurance Study, and the HEC-SSP methodologies are shown in Table 5-1.

Weighting Factors for Peak Discharges Development								
Methodology	Accuracy in Equivalent Years of Record							
Percent Chance Exceedance	50%	20%	10%	5%	2%	1%	0.5%	0.2%
HEC-HMS	20	20	20	20	20	20	20	20
Regression	15	15	15	15	15	16		
Plate 6						10		
FEMA			15 25(JP3)		15 25(JP3)	15 25(JP3)		15 25(JP3)
HEC-SSP	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)	59(JP1) 32 (JP3) 38(JMP2)

Table 5-1. Weighting Factors Used To Develop Final Peak Flow Values

Determination of the final peak flow discharges at junctions of interest for the sub-watersheds studied was conducted in three steps: (1) the peak flow discharge values produced by each method were weighted; (2) all the available peak flow discharge values were plotted on log probabilistic graph paper by percent chance exceedance; and (3) the best fit curve of the peak flow discharges was graphed assuming watershed linearity, that is, that the peak flow discharge-frequency curves should be defined by a single function (illustrated as a smooth curve) for each sub-watershed.

The determination of final peak flow discharges assumes that the sub-watersheds examined in this study exhibit linearity, meaning that a single function may describe the runoff from a sub-watershed. Sub-watershed linearity is based on the concept that peak flow discharge frequency curves serve their descriptive purpose as continuous, smooth curves. Thus, even after peak flow discharges were weighted and plotted on log-probabilistic graph paper, the best curve fit for these discharge values was plotted. The best fit curve was the final step in determining peak flow discharge values at the junctions of interest

5.2 Makiki Peak Flow Discharges

Peak flow discharges at junctions of interest in the Makiki sub-watershed were weighted according to the process detailed in Section 5.1, plotted on log-probabilistic graph paper, and a best fit curve was analyzed. Table 5-2 provides peak flow discharge results for the Makiki sub-watershed at junctions of interest by methodology, the weighted values, and the 'FINAL' best fit values.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JK1 (Confluence of Makiki and Kanaha Streams, A=2.328 mi²)								
HEC-HMS	570	1,200	1,890	2,400	3,150	3,740	4,380	5,240
Regression	660	1,350	1,960	2,900	3,840	4,680		
Plate 6						5,300		
Weighted	610	1,260	1,920	2,620	3,450	4,410	4,380	5,240
FINAL	**650**	**1,300**	**1,900**	**2,550**	**3,400**	**4,100**	**4,800**	**5,700**
JK2 (USGS Stream Gage at King St. 16238000, A= 2.49 mi²)								
HEC-HMS	660	1,360	2,110	2,650	3,440	4,060	4,730	5,630
Regression	670	1,370	2,000	2,980	3,960	4,850		
Plate 6						5,600		
FEMA			1,850		3,250	3,950		5,950
Weighted	660	1,360	2,000	2,790	3,540	4,490	4,730	5,770
FINAL	**660**	**1,330**	**1,960**	**2,580**	**3,500**	**4,250**	**4,950**	**5,900**
JK3 (Confluence of Makiki Stream and Ala Wai Canal, A=2.892 mi²)								
HEC-HMS	890	1,770	2,690	3,340	4,280	5,000	5,790	6,850
Plate 6						6,100		
Weighted	890	1,770	2,690	3,340	4,280	5,370	5,790	6,850
FINAL	**760**	**1,600**	**2,400**	**3,200**	**4,300**	**5,250**	**6,100**	**7,200**

Table 5-2. Peak Flow Discharges at Makiki Junctions by Methodology

The junction near the confluence of the Makiki Stream and Ala Wai Canal (JK3) received the highest amount of peak flow discharge in the Makiki sub-watershed. This was expected, because JK3 represents the flow exiting the entire Makiki sub-watershed. The peak discharge values attained by the Plate 6 and Regression methods appear higher than the peak discharge values attained through HEC-HMS modeling, as seen in Figures 5-1 through 5-3. As mentioned in Section 5.1, the peak discharge values were not only weighted, but also the final values were determined by the best fit curve shown in Figures 5-1 through 5-3. This best fit curve takes into account all of the methods used. In short, the final best fit curve was used to calculate the final peak discharges. Figures 5-1 through 5-3 graph the peak flow discharge by methodology over the percent chance exceedance for Makiki junctions of interest.

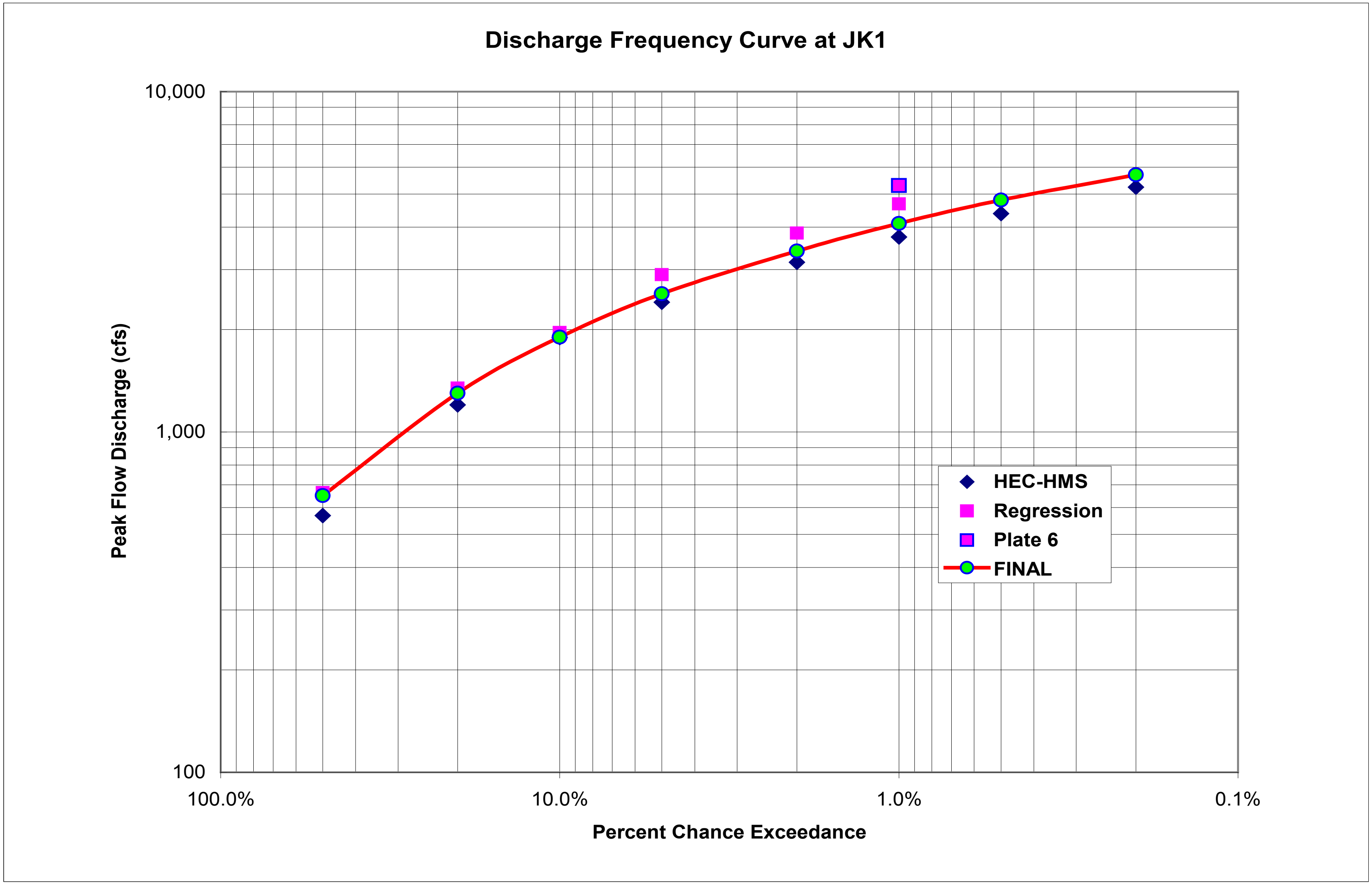

Figure 5-1. Final Discharge Frequency Curve at JK1 (Confluence of Makiki and Kanahā Streams)

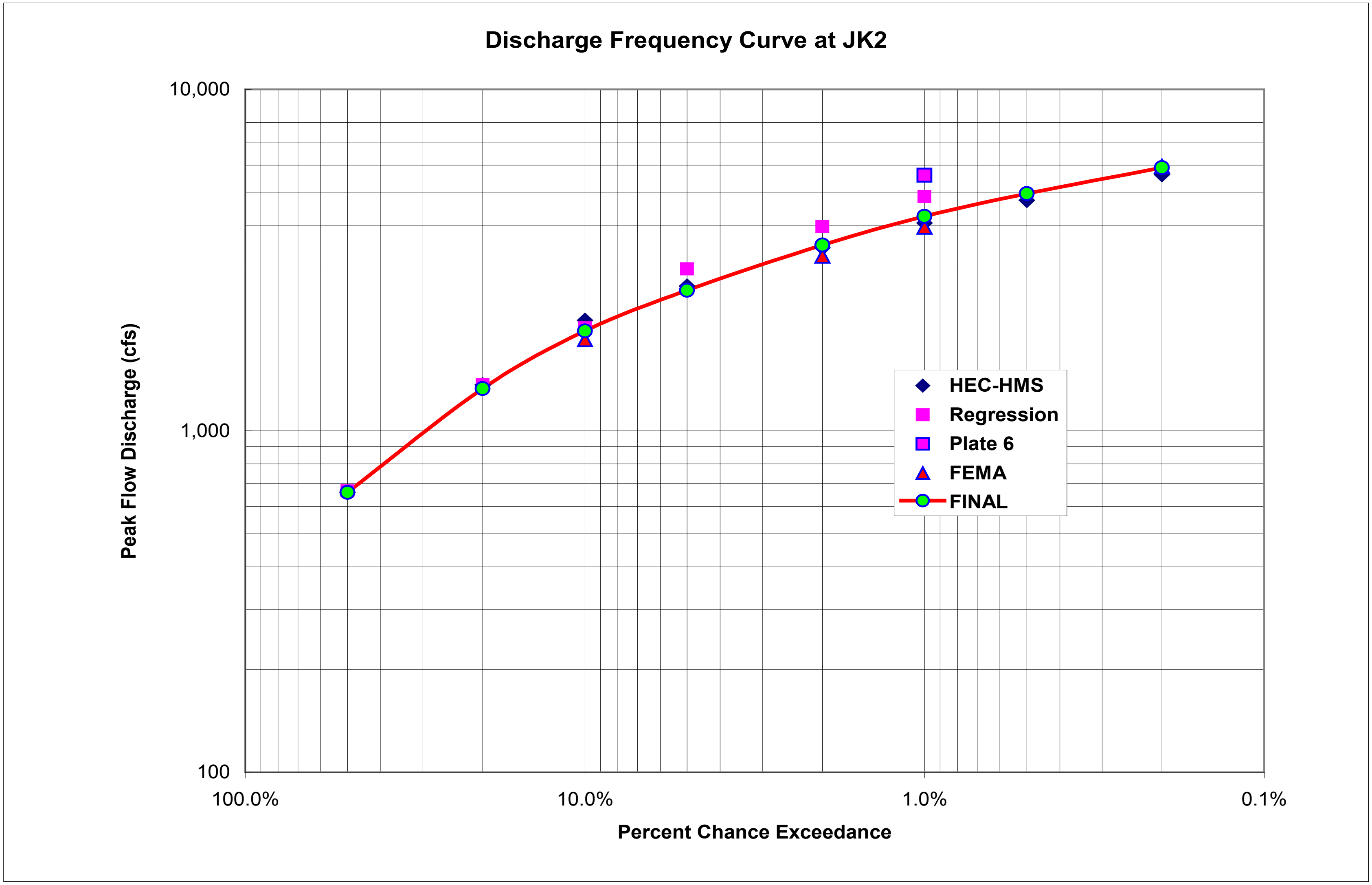

Figure 5-2. Final Discharge Frequency Curve at JK2 (USGS Stream Gage [16238000])

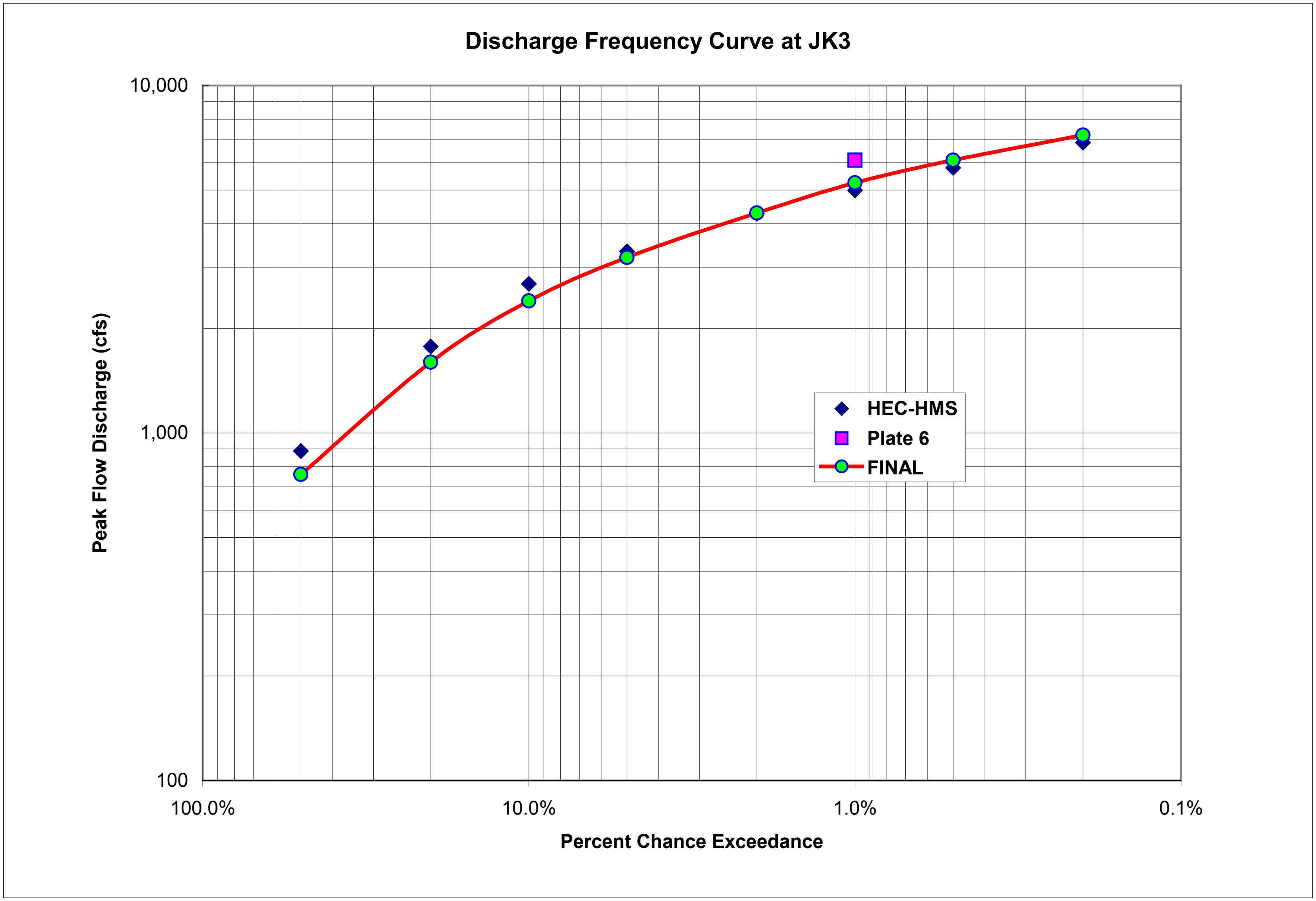

Figure 5-3. Final Discharge Frequency Curve at JK3 (Confluence of Makiki Stream and Ala Wai Canal)

5.3 Mānoa Peak Flow Discharges

Peak flow discharges for the Mānoa sub-watershed were determined in a previous study, and these values were used for the current study. The HEC-HMS peak flow discharges calculated in the Mānoa Watershed Project hydrology report (Oceanit 2008) at the junction just upstream of the confluence of the Mānoa and Pālolo Streams (JM8) were used. This junction, JM8, is where flow exits the Mānoa sub-watershed, and thus this peak discharge value accounts for all the runoff exiting the Mānoa sub-watershed. Table 5-3 provides the peak flow discharge results by methodology and the 'FINAL' values,. The final peak flow discharges from this study are plotted in Figure 5-4.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JM8 (Right above Confluence of Manoa and Palolo Streams, A=5.972 mi^2)								
HEC-HMS	2,560	4,450	6,210	7,860	9,810	11,100	12,400	14,500
Regression	1,660	3,100	4,330	6,120	7,870	9,330		
Plate 6						11,000		
FEMA			7,600		11,500	13,600		17,000
Weighted	2,180	3,870	6,060	7,110	9,730	11,200	12,400	15,600
FINAL	**2,600**	**4,450**	**6,150**	**7,800**	**9,700**	**11,000**	**12,400**	**14,400**

Table 5-3. Peak Flow Discharges at Mānoa Junctions by Methodology

The junction that is just upstream of the confluence of the Mānoa and Pālolo streams (JM8) receives the highest amount of peak flow discharge in the Mānoa sub-watershed. Figure 5-4 illustrates the peak flow discharge results at the Mānoa junctions of interest.

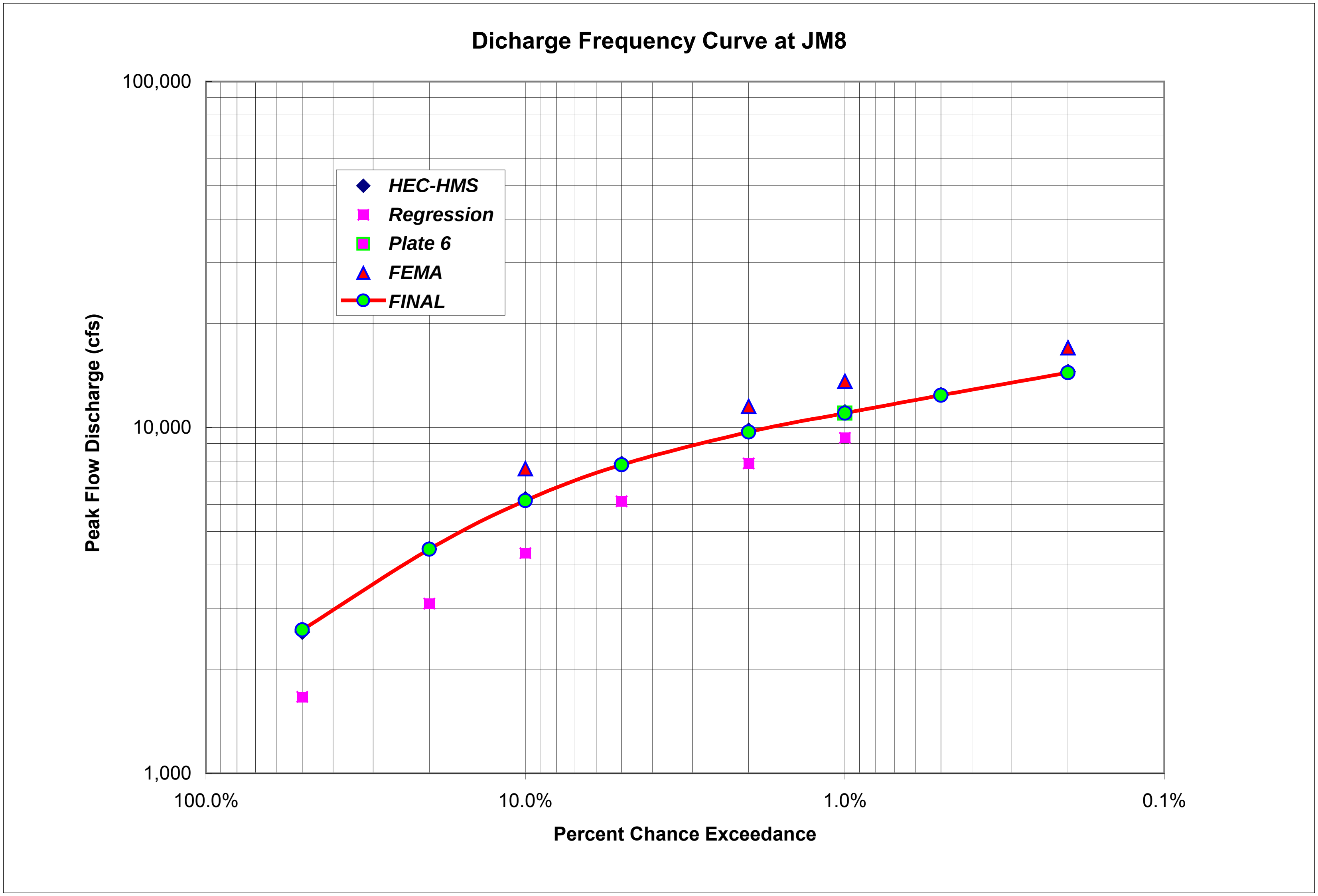

Figure 5-4. Final Discharge Frequency Curve at Junction JM8 (Upstream of the Confluence of Mānoa & Pālolo Streams)

5.4 Pālolo Peak Flow Discharges

Pālolo peak flow discharges at junctions of interest were determined through the process described in Section 5.1. Table 5-4 provides peak flow discharge results for the sub-watershed by methodology and weighted followed by 'FINAL' values.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JP1 (Pukele Stream Gage 16244000, A= 1.146 mi^2)								
HEC-HMS	320	730	1,150	1,460	1,900	2,220	2,590	3,110
Regression	650	1,160	1,580	2,200	2,810	3,320		
Plate 6						3,400		
HEC-SSP	420	810	1,150	1,530	2,110	2,620	3,190	4,050
Weighted	440	850	1,220	1,620	2,180	2,720	3,040	3,810
FINAL	**400**	**800**	**1,150**	**1,550**	**2,100**	**2,500**	**2,900**	**3,400**
JP2 (Confluence of Pukele and Waiomao Streams, A=2.938 mi^2)								
HEC-HMS	940	2,030	3,190	4,010	5,180	6,040	6,980	8,320
Regression	1,035	1,930	2,700	3,828	4,940	5,880		
Plate 6						6,200		
Weighted	980	1,990	2,980	3,930	5,080	6,020	6,980	8,320
FINAL	**950**	**1,850**	**2,700**	**3,650**	**4,900**	**5,900**	**6,900**	**8,000**
JP3 (Palolo Stream Gage 16247000, A=3.62 mi^2)								
HEC-HMS	1,330	2,710	4,170	5,180	6,620	7,670	8,850	10,500
Regression	1,040	2,020	2,870	4,150	5,410	6,500		
Plate 6						7,700		
FEMA			2,790		4,510	5,340		7,530
HEC-SSP	1,210	2,090	2,780	3,510	4,580	5,470	6,430	7,820
Weighted	1,210	2,260	3,100	4,150	5,140	6,240	7,360	8,410
FINAL	**1,200**	**2,100**	**3,000**	**4,000**	**5,500**	**6,500**	**7,500**	**8,600**
JP4 (Right above the confluence of Manoa and Palolo Streams, A= 4.065 mi^2)								
HEC-HMS	1,550	3,120	4,720	5,810	7,400	8,550	9,860	11,600
Regression	1,040	2,060	2,970	4,330	5,690	6,860		
Plate 6						8,100		
Weighted	1,330	2,660	3,970	5,180	6,660	7,870	9,860	11,600
FINAL	**1,250**	**2,200**	**3,100**	**4,200**	**5,700**	**6,900**	**7,900**	**9,100**

Table 5-4. Peak Flow Discharges at Pālolo Junctions by Methodology

The junction that is just upstream the confluence of the Mānoa and Pālolo streams (JP4) receives the highest amount of peak flow discharge in the Pālolo sub-watershed, as it is situated at the downstream (*makai*) end of the watershed and drainage system. In the Pālolo sub-watershed, at the Pūkele Stream gage junction (JP1), the regression method calculates higher flow discharge values than other methods, and the HEC-HMS model seems to underestimate the peak flow discharges for many of the storms under study; the discharge frequency curve fit closely mirrors the findings of the

HEC-SSP analysis which applied 59 historical annual peaks. However, at the next junction downstream, the confluence of the Pūkele and Waiʻōmaʻo Streams (JP2), all the methodologies used provide similar peak flow discharge values. The HEC-SSP analysis was not used for this junction. The discharge frequency curve for the junction at the Pālolo Stream gage (JP3) seems to be higher than HEC-SSP findings at lower exceedance probabilities, this is probably due to the shorter historical annual peak records and the incontinuous and incorrect records. Downstream at the junction just upstream of the confluence of the Mānoa and Pālolo Stream (JP4), the frequency curve fit is close to the low regression equation values. All of these results are illustrated by junction for the Pālolo sub-watershed in Figures 5-5 through 5-8. These figures graph the peak flow discharge by method over the percent chance exceedance storm.

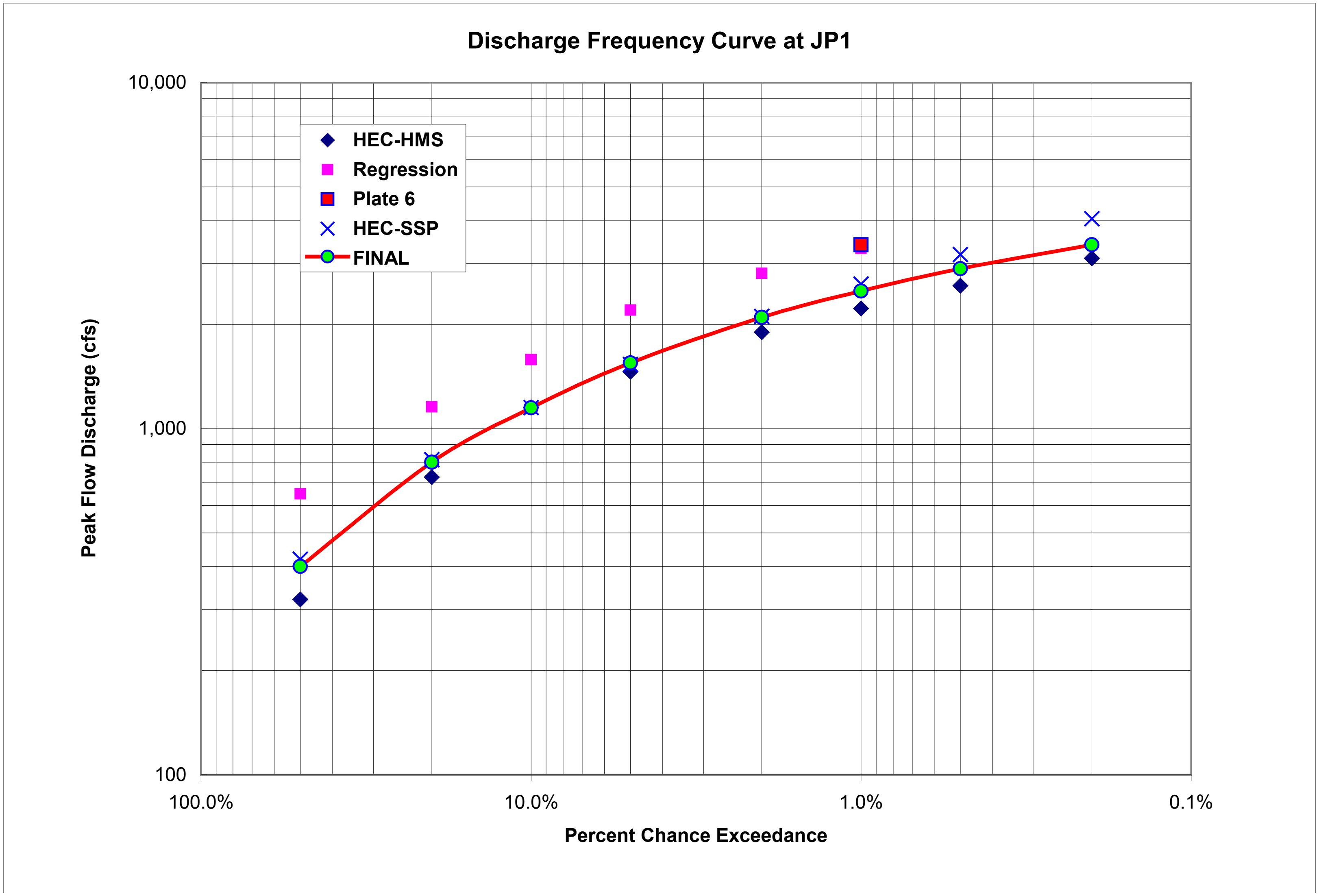

Figure 5-5. Final Discharge Frequency Curve at JP1 (USGS Pūkele Gage [16244000])

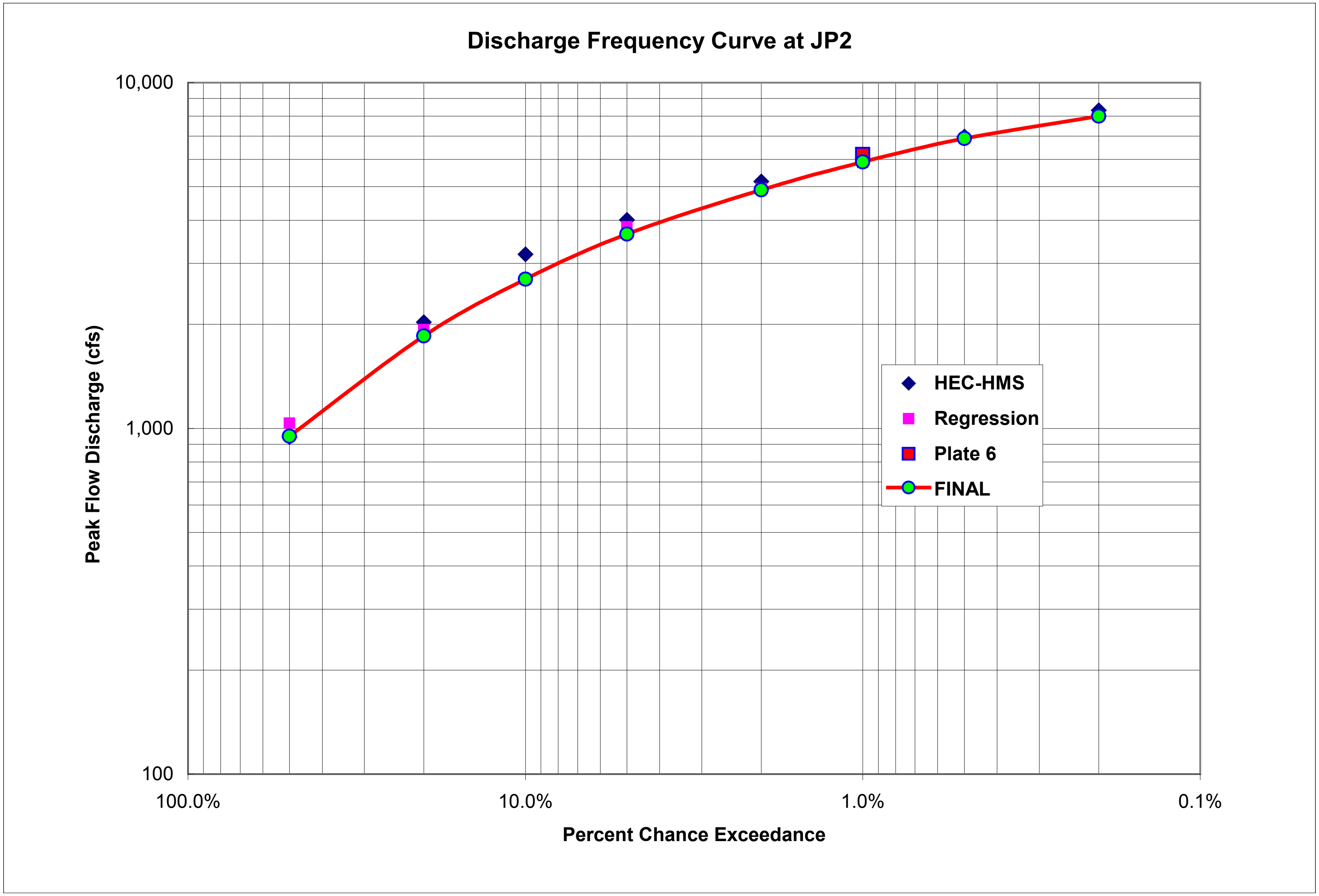

Figure 5-6. Final Discharge Frequency Curve at JP2 (Confluence of Pūkele and Wai'ōma'o Streams)

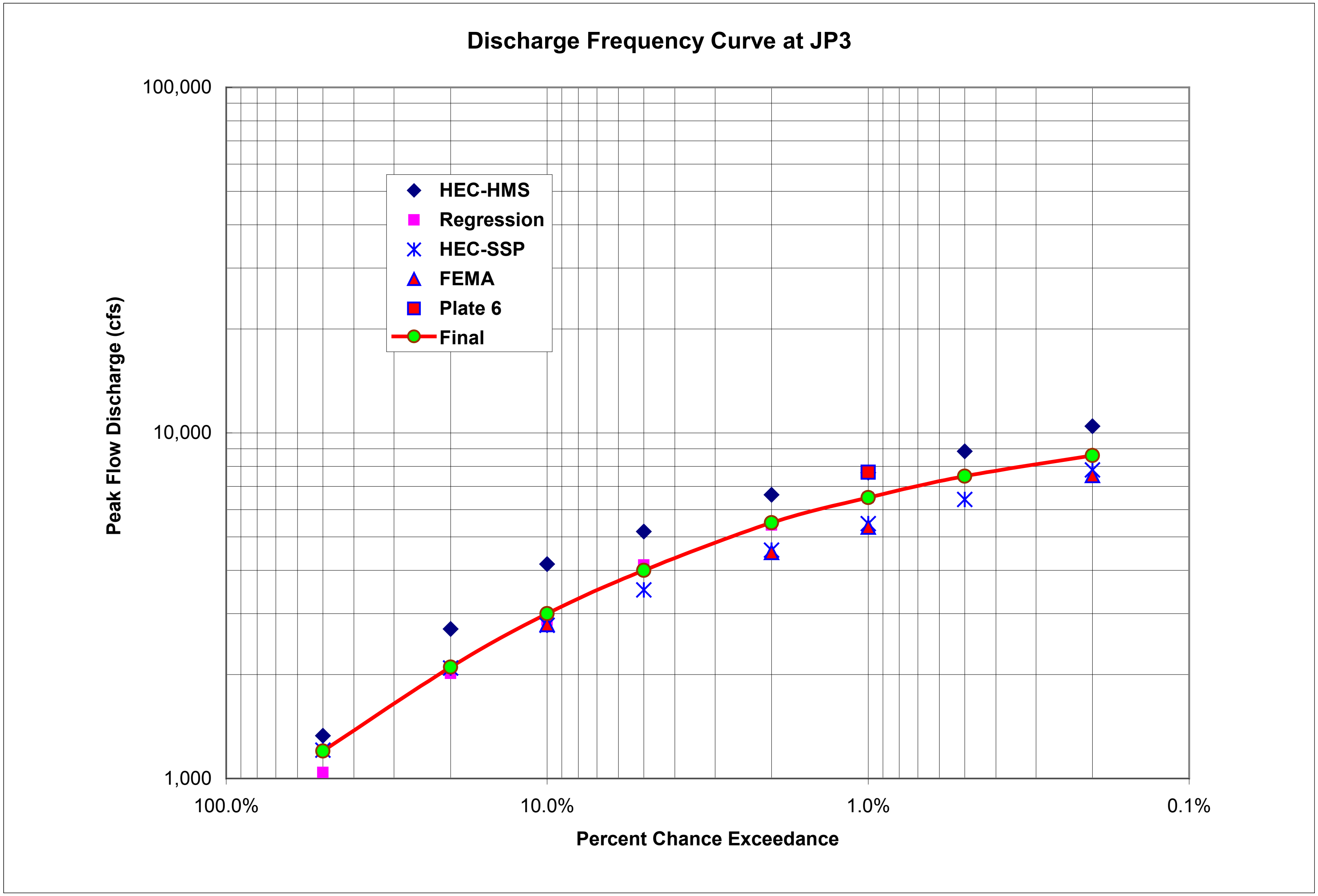

Figure 5-7. Final Discharge Frequency Curve at JP3 (USGS Pālolo Gage [16247000])

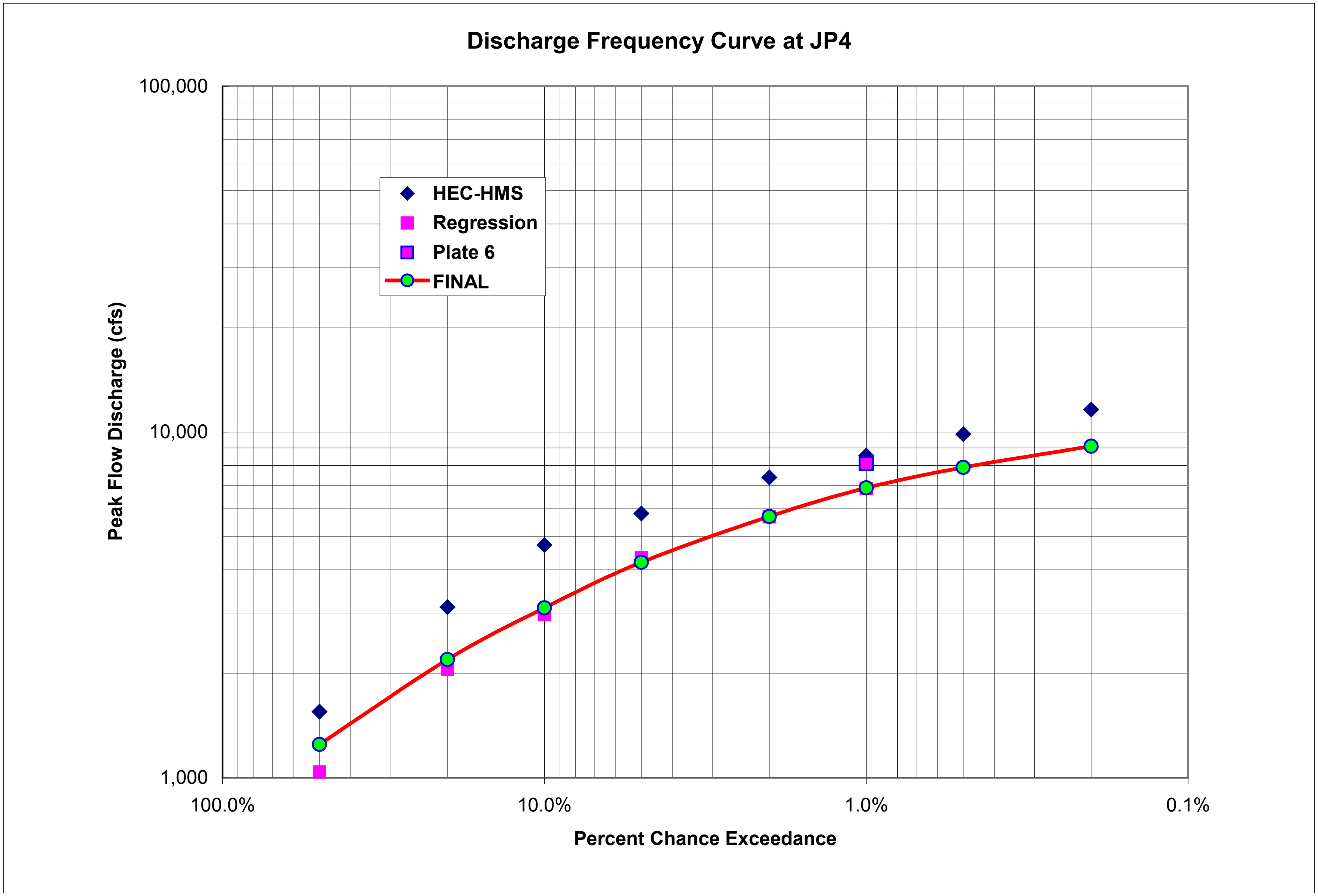

Figure 5-8. Final Discharge Frequency Curve at JP4 (Upstream of the Confluence of Mānoa & Pālolo Streams)

5.5 Mānoa-Pālolo Peak Flow Discharges

Weighting of methodologies were used where peak flow discharges for multiple methodologies were available. Table 5-5 provides peak flow discharge results for the Mānoa-Pālolo Canal by methodology and then as 'FINAL' values through the weighting process described.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
JMP1 (Confluence of Manoa and Palolo Streams, A= 10.037 mi²)								
HEC-HMS	4,020	7,170	10,300	12,900	16,100	18,500	20,900	24,400
Regression	2,120	4,060	5,760	8,240	10,700	12,700		
Plate 6						15,500		
Weighted	3,210	5,840	8,360	10,900	13,800	15,800	20,9000	24,400
FINAL	**3,350**	**6,000**	**8,400**	**10,900**	**14,100**	**16,500**	**18,700**	**21,800**
JMP2 (Manoa-Palolo Stream Gage 16247100, A= 10.34 mi²)								
HEC-HMS	4,090	7,340	10,500	13,000	16,300	18,700	21,100	24,700
Regression	2,110	4,080	5,800	8,320	10,800	12,900		
Plate 6						16,000		
FEMA			12,000		19,200	23,000		28,500
HEC-SSP	2,883	5,065	6,800	8,670	11,400	13,700	16,200	19,800
Weighted	3,070	5,520	8,470	9,890	13,900	16,400	18,100	23,200
FINAL	**3,400**	**6,100**	**8,500**	**11,150**	**14,400**	**16,800**	**19,000**	**22,100**
JMP3 (Right above the confluence of Manoa-Palolo and Ala Wai Canals, A=10.678 mi²)								
HEC-HMS	4,220	7,450	10,700	13,300	16,600	18,900	21,400	24,900
Plate 6						16,500		
Weighted	4,220	7,450	10,660	13,260	16,560	18,100	21,400	24,900
FINAL	**3,450**	**6,200**	**8,700**	**11,400**	**14,700**	**17,100**	**19,300**	**22,400**

Table 5-5. Peak Flow Discharges at Mānoa-Pālolo Junctions by Methodology

The junction directly upstream of the confluence of the Mānoa-Pālolo and Ala Wai Canals (JMP3) receives the highest amount of peak flow discharge in the Mānoa-Pālolo Canal sub-watershed. This junction is located at the downstream (*makai*) end of the watershed and drainage system, and so it is not surprising that peak flow discharge would occur at the 'bottom' of the sub-watershed as the water flows down toward sea level. For the Mānoa-Pālolo Canal junctions studied (JMP1 and JMP3), the HEC-HMS modeling results provide higher peak flow discharges than the other methodologies used, particularly the Regression method and HEC-SSP calculation. At junction JMP2 (USGS gage 16247100), the final *best* estimates are lower than HEC-SSP findings but parallel to those values. Noda and Associates (1994) used 24 historical annual peaks to determine the peak flow discharges; their result for 100 year was at 12,429 cfs, whereas in this study, HEC-SSP provided 13,700 cfs. These results are illustrated in the final discharge frequency curves Figures 5-9 through 5-11. These figures show the peak flow discharge by method and junction, and dependent on the percent chance exceedance storm.

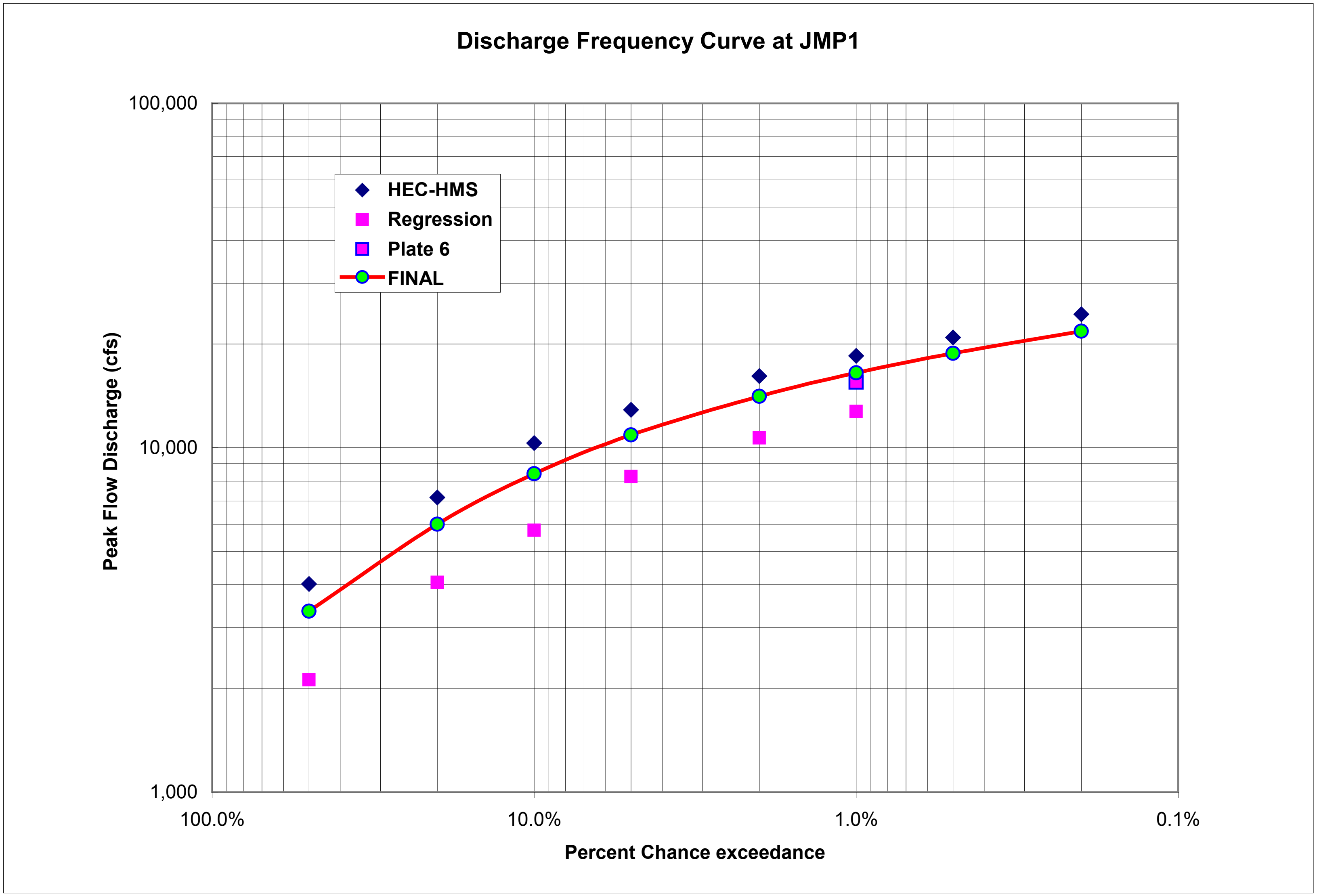

Figure 5-9. Final Discharge Frequency Curve at JMP1 (Confluence of Mānoa & Pālolo Streams)

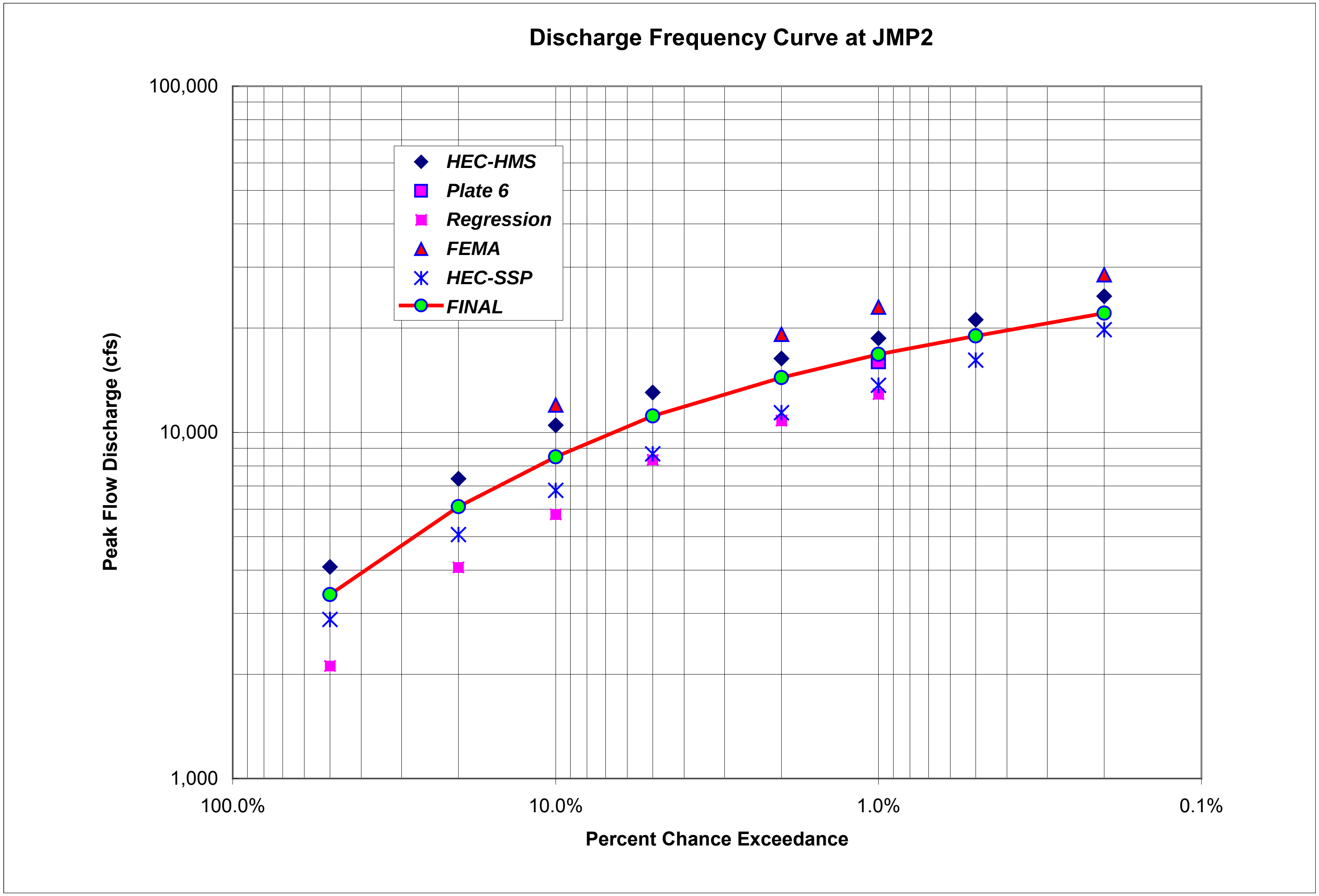

Figure 5-10. Final Discharge Frequency Curve at JMP2 (USGS Stream Gage [16247100])

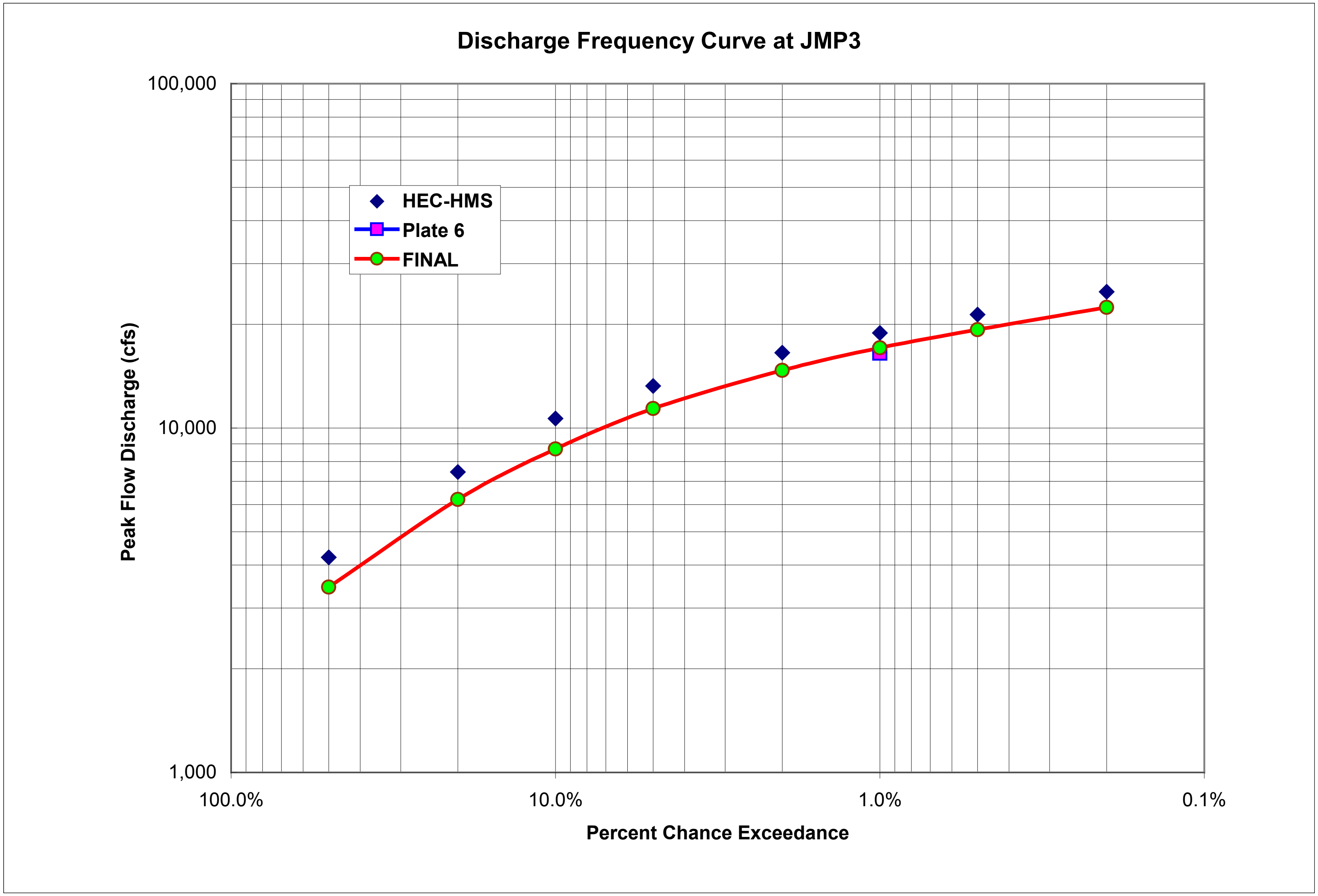

Figure 5-11. Final Discharge Frequency Curve at JMP3 (Confluence of Mānoa -Pālolo and Ala Wai Canals)

5.6 Ala Wai Canal Peak Flow Discharges

As mentioned earlier, Ala Wai Canal was modeled as a reservoir, considering backwater effects caused by the tides due to the sub-watershed location near mean sea level. The reservoir model treated Ala Wai Canal and the adjacent lower area as a detention basin. As the modeled flood wave passes through the reservoir, storage occurs that can greatly reduce the peak flow. The magnitude of this reduction depends on the boundary setting of the modeled reservoir. The storage-elevation function for the Ala Wai Canal reservoir model was determined using bathymetric survey data for the channel and LiDAR data for the surrounding area (Section 4.6.1). No other method accounted for analysis of the surrounding storage area; consequently, the flow peaks determined by other methods are much higher than those determined by the reservoir model. In conclusion, the HEC-HMS results that modeled Ala Wai Canal as a reservoir are considered the most accurate.

Table 5-6 provides peak flow discharge results for Ala Wai Canal sub-watersheds by methodology and then weighted followed by 'FINAL' values through the best fit curve process.

Methodology	Peak flow discharge (cfs)							
Return Period (yr)	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Percent Chance Exceedance	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
Ala Wai Canal (Mouth of Ala Wai Canal, A=16.215 mi^2)								
HEC-HMS	6,000	10,100	13,400	15,200	16,700	17,700	18,700	20,500
Plate 6						22,500		
FEMA			13,700		23,000	28,200		36,200
Weighted	6,000	10,100	13,500	15,200	19,400	22,300	18,700	27,200
FINAL	**6,000**	**9,500**	**12,500**	**15,200**	**17,500**	**18,500**	**19,500**	**20,500**

Table 5-6. Peak Flow Discharges at the Ala Wai Canal Mouth by Methodology

The inflows to Ala Wai Canal increased, whereas the outflow did not increase significantly. For example, at the 50-year frequency storm, inflow was estimated as 24,850 cfs from HEC-HMS model, and the outflow from the Ala Wai Canal was estimated as 16,700 cfs with a peak elevation of 5.4 feet. At the 100-year frequency storm, HMS model shows that inflow was 28,200 cfs, and outflow was 17,700 cfs at a peak elevation at 5.8 feet. The canal will be overtopped at this storm condition and the water will be stored in the adjacent areas. Figure 5-12 shows the peak flow discharge over the percent chance exceedance by methodology at the mouth of the Ala Wai Canal.

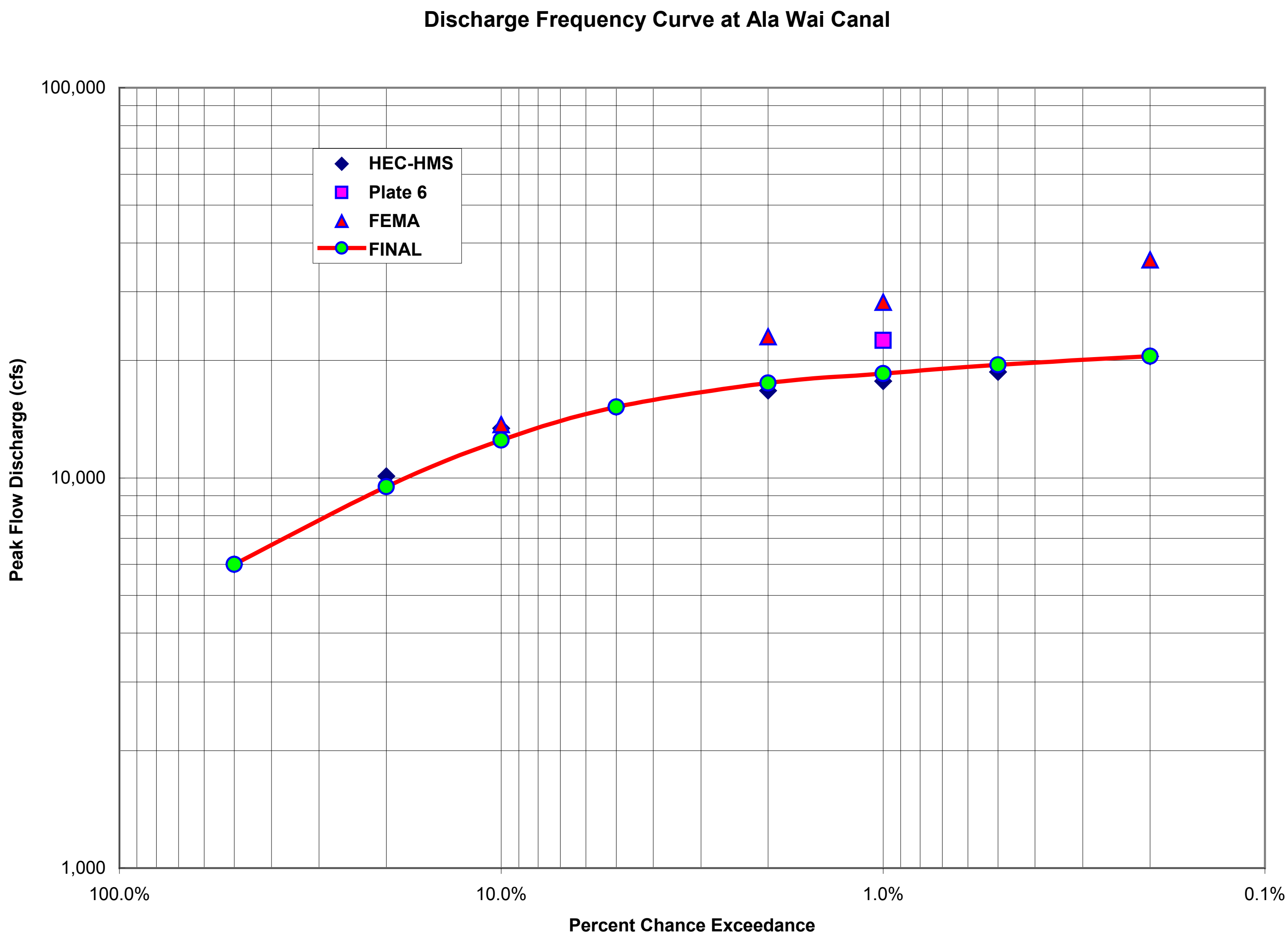

Figure 5-12. Final Discharge Frequency Curve at the Mouth of the Ala Wai Canal

5.7 Peak Flow Discharge Update (November 2010)

As discussed in Hydraulic Appendix, peak flow values were updated and adjusted based on new rainfall-frequency-intensity data. When the hydrologic studies for Manoa and Ala Wai Watersheds were conducted, the 1984 rainfall frequency data for Oahu was used in the rainfall-runoff modeling (Giambelluca and others, 1984). In March 2009, the updated rainfall frequency data for the State of Hawaii was released as the Precipitation Frequency Data Server (PFDS) which is part of National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 4, Version 2.0, Hawaiian Islands, released March 30, 2009. Atlas 14 is official documentation of precipitation frequency estimates for the United States. Documentation can be found at: http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume4.pdf, last accessed September 28, 2009 while the actual; server is located at: http://hdsc.nws.noaa.gov/hdsc/pfds/hi/hi_pfds.html. This tool computes the rainfall frequency and intensity with 90 percent confidence limits for the 1- to 100-year storms for durations from 5 minutes to 60 days. The updated rainfall frequency values are presented in Table 5-7. A comparison between the previous (Table 4-1) and newer rainfall frequency duration values, indicated that the newer intensity values were higher than the older data by an average of 4 to 13 percent depending on the rainfall recurrence interval and duration. In general, the shorter frequency time periods had a larger change then the longer rainfall time periods. To provide consistency with using the HEC-RAS model for the future without project and alternative modeling, the peak-flow data was updated to account for the impacts of the new higher rainfall frequency-duration data for the Ala Wai Watershed. The peak-flow input data was changed from the previous computed input flow data from both the Manoa and Ala Wai hydrologic studies by an average of 9.8 percent. The average percent range varied from minus 7 percent for the Manoa Stream 10-percent chance flood to plus 36 percent for the non-Manoa Stream 50-percent chance floods (Table 5-8). Because the peak flow data was not solely based on the HEC-HMS rainfall-runoff modeling but also other methods such as flood-frequency analysis, the peak flow adjustments were not just based on modifying the previous HEC-HMS results but also incorporating the graphical adjustments used in this and the previous Manoa Stream study (Oceanit, 2008b).

The updated adjusted peak discharge values by junction are listed in Table 5-8. These values were then adjusted by location, as described in the Hydraulic appendix for use in the HEC-RAS model. The uncertainty of the peak flow discharge values, as discussed in Section 5.1, is based on the equivalent years of record. The final equivalent years of record (EYOR) used in the risk and uncertainty HEC-FDA model is based on stream reach and is presented in Table 5-9. The Makiki Watershed with the least amount of available data was given the lowest EYOR of 18 years, while the remaining sub-watersheds were assigned values from 25 to 30 years. The highest values were from sub-basins where the peak flow discharges were almost entirely based on gaged data; Pukele and Waiomao Streams.

Percent Chance Exceedance	Return Period (years)	Depth (inches) for Specified Duration								
		5-min	15-min	30-min	1-hour	2-hours	3-hours	6-hours	12-hours	24-hours
99	1	0.38	0.66	0.97	1.40	1.87	2.12	2.74	3.35	3.92
50	2	0.47	0.80	1.19	1.72	2.33	2.71	3.49	4.29	5.18
20	5	0.61	1.04	1.54	2.22	3.04	3.54	4.58	5.68	6.96
10	10	0.72	1.24	1.83	2.64	3.61	4.21	5.46	6.80	8.39
5	20	0.81	1.49	2.11	3.05	4.15	4.94	6.28	8.00	9.95
4	25	0.89	1.52	2.25	3.24	4.42	5.16	6.69	8.36	10.42
2	50	1.02	1.75	2.59	3.74	5.09	5.94	7.69	9.61	12.05
1	100	1.16	1.99	2.95	4.25	5.78	6.74	8.74	10.92	13.77
0.5	200	1.31	2.25	3.34	4.82	6.53	7.61	9.86	12.30	15.60
0.2	500	1.53	2.62	3.88	5.61	7.57	8.82	11.42	14.23	18.18

Rainfall Intensity Frequency data determined from NOAA Atlas 14 Precipitation Frequency Data Server using watershed centroid of 21.3092 N, 157.8071 W. Values for the 5-percent chance storm are interpolated.

Revision of data in Table 4-1

Table 5-7. Updated Rainfall Intensity Frequency Data for the Ala Wai Watershed, Oahu, Hawaii

Model & Junction	Return Period (yr)	2	5	10	20	50	100	200	500
	Percent Chance Exceedance	50%	20%	10%	5%	2%	1%	0.50%	0.20%
Manoa Watershed Model	JM1	1,200	2,000	2,600	3,350	4,500	5,400	6,200	7,600
	JM2	1,940	3,200	4,200	5,280	7,140	8,350	9,400	11,400
	JM3	2,080	3,450	4,350	5,450	7,200	8,410	9,500	11,600
	JM4	2,200	3,650	4,600	5,700	7,500	8,700	10,000	12,500
	JM5	2,320	3,800	4,800	6,100	7,900	9,360	11,000	12,900
	JM6	2,500	4,100	5,200	6,530	8,800	10,200	12,000	14,200
	JM7	2,700	4,300	5,600	6,900	9,250	10,700	13,000	15,000
	JM8	2,900	4,600	6,100	8,200	10,400	12,500	14,500	17,400
Ala Wai Watershed Model	JK1	800	1,500	2,100	2,770	3,800	4,700	5,500	6,600
	JK2	900	1,550	2,200	2,800	3,850	4,800	5,600	6,700
	JK3	1,040	1,850	2,600	3,400	4,700	5,900	6,800	8,500
	JP1	440	850	1,200	1,600	2,280	2,800	3,350	4,200
	JP2	1,100	2,100	3,000	3,930	5,430	6,700	8,020	9,990
	JP3	1,350	2,420	3,400	4,340	5,900	7,420	9,000	11,000
	JP4	1,450	2,580	3,500	4,560	6,350	7,900	9,400	12,000
	JMP1	4,200	7,100	9,200	12,000	16,000	18,500	22,100	26,500
	JMP2	4,500	7,300	9,500	12,400	16,200	19,400	22,500	26,900
	JMP3	4,600	7,350	9,700	12,800	16,500	20,000	23,000	27,700
	Ala Wai	8,000	11,500	13,500	16,000	18,000	19,500	20,500	22,000

Table 5-8. Updated Peak Flow Discharges for the Ala Wai Watershed by HEC-HMS Model Junction

Table 5-9. Peak Flow Discharge Frequency Data and Uncertainty in Equivalent Years of Record used in HEC-FDA, Ala Wai Watershed, Oahu, Hawaii

Stream or Sub-Watershed	HEC-HMS Model Sub-Basin or Junction	HEC-RAS Reach Name	HEC-FDA Reach Name	HEC-FDA Analytical Frequency Curve Data (Log Units)			EYOR
				Mean	Std. Dev.	Skew	
Ala Wai, Waikiki	Ala Wai	Ala Wai Lower	ALA 1	3.7983	0.3143	-2.2259	30
		Ala Wai Middle	ALA 2	3.6600	0.2052	0.1873	
		Ala Wai Upper	ALA 3	2.9714	0.2164	-0.7680	
Makiki	K2	Kanaha Ditch	KAH 1 KAH 2	2.4673	0.2954	0.106	18
	----	Kanaha Split	KAO 1	2.2952	0.3480	-0.0938	
	JK3	Makiki Lower	MAK 1	2.9345	0.1638	1.2305	
	JK2		MAK 2	2.8820	0.1609	1.7006	
	JK1	Makiki Upper	MAK 3	2.6086	0.2634	0.2515	
	K1, K3		MAK 4	2.3121	0.3323	0.1887	
Manoa	JM7, JM 8	Manoa Stream Main Reach	MAN 1	3.4780	0.2340	0.4426	25
	JM 6		MAN 2	3.3770	0.2299	0.2732	
	JM 4, JM 5		MAN 3 MAN 4	3.3297	0.2339	0.2878	
	JM 3		MAN 5	3.3000	0.2444	0.2758	
	JM 1, JM 2		MAN 6 MAN 7	3.0954	0.2436	0.4493	
	----	UH_Split	UNI 1 UNI 2	0.699	0.7764	0.0153	18
Manoa-Palolo Canal	JMP 1 to JMP 3	Palolo Lower	MPC 1 MPC 2	3.6356	0.2482	0.280	30
Palolo	JP1 to JP 4	Palolo Main	PAL 1 to PAL 4	3.1354	0.3063	0.136	27
	JP1	Pukele Tributary	PUK 1	2.8005	0.3424	-0.057	44
	P2, P5	Waiomao Ditch	WAI 1	2.8129	0.2976	-0.021	35

EYOR = Equivalent Years of Record; ----, not a separate sub-basin in HEC-HMS model

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